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Economy of Steel Trussed Roofs

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ECONOMY OF STEEL TRUSSED ROOFS

BY

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This is to certify that the thesis of LEWIS LAWRENCE POWELL entitled ECONOMY OF STEEL TRUSSED ROOFS was prepared under my personal supervision; and I recommend that it be approved as meeting this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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THE ECONOMY OF STEEL TRUSSED ROOFS.

I. INTRODUCTION.

Art. 1. Previous Theses.

In June, 1909, Messrs. G. M. A. Ilg, and R. C. Wagner, Jr. submitted a thesis to the University of Illinois, on this same subject, stating that they had chosen to investigate this problem as it was one which had seemingly been neglected as contrasted with the elaborate studies of the economy of many other engineering structures.

After discussion of those factors entering into the question, they designed trusses of the most economic of the various types, of 40, 60, 80, and 100-foot spans, with the most commonly used pitches, i. e., 1/5, 1/4, 30°, and 1/3. Keeping the spacing constant, 16 feet, they drew curves showing the variation of weight with pitch, and in a like manner investigated the most economical spacing of trusses for weight only, and for actual cost. In addition to this, they plotted the most common formulas for truss weights for varying spans, and derived a formula of their own. Their results will be found among the graphs, on pages 34 to 37, 40 and 41, and 43.

On the same date, June 1909, Mr. C. E. Noerenberg presented a thesis entitled, "Economic Design of Steel Roofs". This was submitted in partial fulfillment of the requirements for the degree of Architectural Engineer, while the thesis of Messrs. Ilg and Wagner was for the degree of Bachelor of Science.

As might be expected, Mr. Noerenberg made a more thorough study than the latter, though along somewhat different lines, embracing a wide variety of trusses and compiling a large volume of references in regard to loads, etc.

So far as general assumptions regarding loads and other matters connected with the problem are concerned, frequent reference has been made to these theses, particularly to that of Mr. Noerenberg, to whom the author is especially indebted.

Art. 2. Purpose of this Thesis.

While Messrs. Ilg and Wagner found curve points for the most common pitches and truss spacings, there is, of course, doubt as to the exact locations of the curves for points in between. That they recognized that there was still work to be done in this line is shown by a quotation from Messrs. Ilg and Wagner's thesis:- "This study, investigated in detail, was found to be in a field of almost unlimited extent".

Therefore, the author of this thesis proposes to discuss the factors governing the economy of steel trussed roofs, and continue the investigation commenced by the authors of the work mentioned, by a study of trusses "for points in between", thus establishing more nearly the laws of economic arrangement.

Fowler's "General Specifications for Steel Roofs and Buildings" and Ketchum's "General Specifications for Steel Framed Mill Building" have served as references in the designing of the trusses.

II. ASSUMED LOADINGS.

Art. 3. Roof Loads.

The materials in use for roof coverings are, Corrugated Steel, slate, tile, tin, sheet steel, gravel, slag, asphalt, shingles, asbestos, granite, and numerous patent roofings. Of these, all, except a few of the patent roofings and corrugated steel, are of necessity, laid on sheathing. The corrugated steel and similar patent roofings may be laid either on sheathing or directly on the purlins.

The choice of a roof covering will depend upon conditions of any one case. The following abstract presented before the Association of Railway Superintendents of Bridges and Buildings in 1902, gives an idea of good practice - or what was good practice ten years ago.

"Slate is much used for station buildings when there is not much climbing for repairs, skylights, and telegraph wires. It has a life of from 35 to 40 years, and the roof should have a pitch of not less than 6 inches per foot. Vitrified tile is very desirable when rightly made, and laid on steep roofs, but is not adapted for ordinary railroad buildings. Shingle roofs last as long as 28 years, and should be laid with 6 inches pitch per foot; they are very satisfactory when slate is not too expensive. For flat roofs, a tar and gravel composition is preferred, and will last 12 to 18, and even 20 years.---------------Asphalt pitch is sometimes preferred to coal tar, but the latter is sufficiently durable. An Asphalt gravel roof must slope not more than 1/2 inch to the foot, on account of the liability to run in hot weather; but tar-gravel roofs may have a pitch of one inch per foot. With such very flat roofs

as are required for asphalt, any settlement will form hollows that will hold water".

"Sheet metal roofs, corrugated or flat, are not durable. Steel is less durable than iron, and will last only about one year when exposed to engine gases. Tin shingles of good quality will give good results. Painted shingles have a short life unless frequently painted".

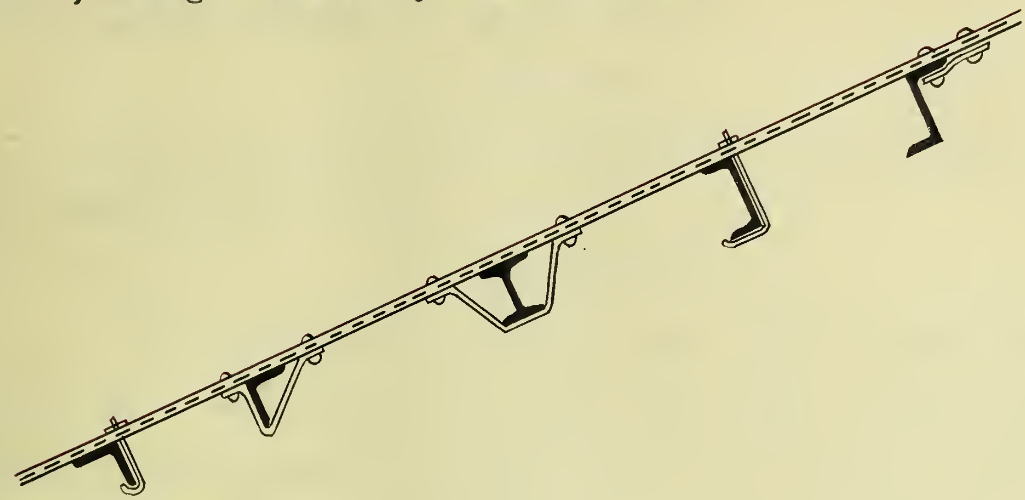
On the other hand, to quote from Ketchums "Steel Mill Buildings",

"An engine house with anti-condensation roofing", (ordinary corrugated steel roofing laid over asbestos and tar paper, placed upon galvanized wire poultry nettings to prevent condensation of moisture on inner surface of roof) "has been used in the Lake Superior Copper country for several years, and has been altogether satisfactory under trying conditions". He also quotes another roof of the same sort in East Helena, Montana, which has given satisfaction, after several years, In each of these cases, the roofs are subject to gases - as their location might indicate.

Corrugated steel has a wide use with some designers, and seems to have given satisfaction under proper conditions. The qualities recommending the use of corrugated steel are its low first cost, and the ease with which it is placed on the roof. It may be gotten in stock lengths from the mills in lengths from 5 to 10 feet, with a width of sheet of 26 inches. Standard corrugations are 2 1/2 inches wide, and 5/8 inches deep, with a thickness of metal varying from 0.0625 to 0.0156 inches.

When placed on the purlins, a common means of fastening is by use of an iron strap placed underneath, and running under the

purlin, being fastened by a small rivet.



Methods of Fastening Corrugated Steel.

In the choice of a roof covering, as well as other factors in design, the author of this thesis, in order that he may directly supplement Messrs. Ilg and Wagner's work, must be governed by their selections; and so corrugated steel will be used here, and with the same thickness as used by them, i. e., 0.0625 inches. In accordance also, with their usuage, the steel will be placed directly upon the purlins, in which case the spacing of the purlins will be determined by the maximum safe unsupported length of corrugated steel.

There have been several formulas proposed for this safe length, of which the following are in common use. The same notation is used throughout:-

Let W = total safe load in lbs.
S = working stress (Ketchum uses 12000 lbs. per sq. in.)
h = depth of corrugation in inches
b = width of sheet in inches
t = thickness in inches
l = clear span in inches

(a) Rankine's formula.

Taking the corrugation curve as a cycloid, he derived the moment of inertia of the section, $I = 2/15(bh^2t)$, and substituting

in the formula $M = SI/C$, (the common flexure formula for homogeneous beams) he derived the following

$$W = 32/15(Shbt/l)$$

If W = safe load in lbs. per square foot of roof, then $W = wbL/144$, and substitution in above, gives

$$w = 307(Sht/L^2)$$

It is specified by Ketchum that purlins should commonly be spaced for a safe load of 30 pounds per square foot of roof, a greater spacing making the roof unsafe to walk upon, and also more liable to leaks.

Using the thickness 0.0625 inches in the formula $L^2 = 307(Sht/w)$, then

$$L^2 = \frac{307 \times 12000 \times 5 \times 1}{8 \times 30 \times 16}$$

$$= 4796.9$$

$$L = 69.27 \text{ inches} = 5.77 \text{ feet}$$

(b) Gage's formula.

By actual and carefully conducted experiments under the direction of Professor Ketchum, Mr. Gage found $W = C \times 8 \times (Shbt/L)$, when C is a constant with the following values, depending upon inclination of metal to horizontal axis;

Angle	C
30°	0.278
45°	0.293
60°	0.312
90°	0.393

Since the spacing would vary with the pitch of the roof, no general spacing will be derived here.

(c) Formula from Dufour's, "Bridge Engineering and Roof Trusses".

$$w = 330(Sht/L^2)$$

This formula was derived by assuming the curve of corrugation to be circular, and it is evident that this will give a slightly larger allowable span than Rankine's formula.

Ilg and Wagner found by Rankine's formula that the maximum allowable span was 5.87 feet instead of 5.77 feet as found in the preceeding calculations. Their calculations are not entirely clear, and Ketchum's curves in his "Mill Buildings", which are based on Rankine's formula, show a maximum allowable span of about 5.73 feet. This set of curves is on a small scale, but at least they show a spacing nearer to 5.77 feet than 5.87 feet.

However, again in accordance with Ilg and Wagner's thesis, a maximum spacing of 5.87 feet will be used.

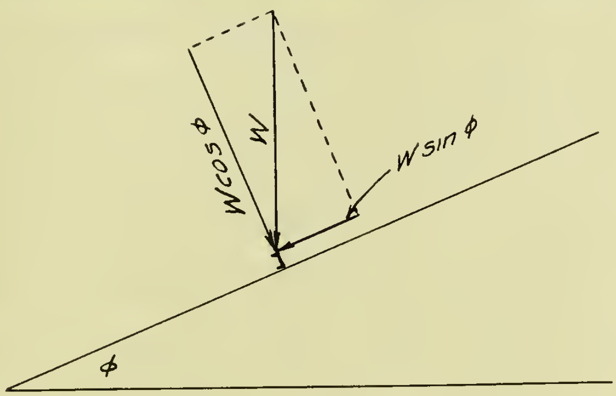
The weight per square foot of number of 16 roofing (thickness = 0.0625") is 3.30 pounds per square foot.

Art. 4. Design of Purlins.

Purlins are members laid on top the trusses at the panel points, running perpendicular to the trusses. They carry the sheathing; or in case sheathing is omitted, the roof is laid direct upon the purlins. The shapes in common use are channels, angles, I-beams, and Z-bars. Of these, the latter is the best adapted, since when laid on a sloping roof, its section modulus increases for a time with the slope of the roof, while for other slopes it decreases constantly. That is, the effect of inclining any purlin shape except a Z-bar, is to weaken it, while with the Z-bar, the slope of the roof has much less influence, the strength in fact, up to a certain point increasing with the pitch. It should be

placed with the leg resting on the roof, pointing downward, as greater stiffness is thus developed. On the contrary, with angles used as purlins, the flange should point up the roof. An increase in the weight of a purlin will generally be more economical than a trussed purlin, as the extra shop expense in a trussed purlin must be considered. When the spacing of trusses is greater than 15 or 16 feet, 3/8 to 1/2 inch tie rods are usually run from the center of one purlin to another. These tie rods take up the component of the loads parallel to the roof, and reduce the moment considerably.

The forces acting on a purlin are vertical (dead load) and normal (component of the wind). If an attempt is made to design the purlin for the resultant of these forces, rather difficult computations are encountered. The same result is accomplished in a much simpler manner by designing for these loads separately.



If W be the total dead load on a purlin, (acting vertically) and ϕ be the angle of inclination of the normal to the roof, with the vertical, (identical with the angle of inclination of the roof to the horizontal), then the normal component of W = W cos ϕ, and the tangential component = W sin ϕ. Taking the normal force, a

section may be approximated by the formula $I/C = M/S$, where $M = WL/8$. Now since the section must resist not only the normal, but also the tangential load, so with this approximate section, using the above formula of $S = MC/I$, (with the weight of the purlin itself now included), the stress due to the normal and tangential loads may be separately calculated. If their algebraic sum does not exceed the specified unit stress, the section is safe as regards the total dead load. If their sum is in excess of the specifications, than another section must be chosen and recalculated.

After a satisfactory section has been found by the above requirements, it must be investigated for wind. The load in this case is the portion of the roof carried by one purlin, plus the weight of the purlin itself, plus the wind load (acting normally). If the maximum fiber stress as calculated with this load is not in excess of the specifications, the purlin is considered safe, since it is nearly improbable that the maximum snow and wind loads will ever occur simultaneously, - and the only load common to each case,- the roof and purling weight, has been included in both calculations.

Even though the rarest case might produce a very heavy snow and ice load, together with a high wind load, accepted engineering economy does not permit design to take into account the very heaviest loads possible under the most extreme conditions, except where safety must be had under all conditions at any cost.

If tie rods are used, the moment of the load is but one-fourth of that without the rods, as the moment varies as the square of the length; and as the length with a tie rod is but one half that without the rod, the moment is therefore reduced to one-fourth its original value.

The wind bracing on the trusses may be considered as a part of the purlin system. Theoretical design is a very difficult matter, and so judgment is usually used in the selection of the proper sections. Bracing will weigh probably not over 1/3 to 3/4 pounds per square foot and this weight is included in the weight of the truss itself, as the assumed weight of the truss is not so close but that this practice may be safely followed.

Art. 5. Snow Load.

In the matter of proper allowance for snow, authorities differ, and the results vary considerably. Various values suggested are 12 to 15 pounds per square foot of roof surface, regardless of slope; 20 to 30 pounds per square foot of horizontal projection; tabulated values for variable pitches, in pounds per square foot of horizontal projection or in pounds per square foot of roof surface; and several values for snow and wind combined. The final choice will depend primarily, of course, on the geographical latitude of the place, and secondarily on the judgment of the designer.

The weight of a cubic foot of snow is quite variable. Freshly fallen snow may weigh 5 to 12 pounds per cubic foot, and snow and hail, or sleet and ice may weigh as high as 30 to 50 pounds, but as the quantity will be quite small even the higher values will not bring a very heavy load upon the truss. To give any one general value for one locality, it is undoubtedly best to state it in pounds per square foot of horizontal projection. Then tables and diagrams for varying pitches will not be necessar, and the calculations will be somewhat simplified.

A common usage specifies twenty pounds per square foot of

horizontal projection and this is advised in all cases but those in exceptional localities, i. e. where the latitude is high or where the humidity is great. This value is that specified by the building laws of Chicago and New York, also by many prominent railroads. This value has also been shown to be the ususal maximum amount of snow load per square foot of horizontal projection for a one-fourth pitch roof in a latitude of 40° to 42° North.

In agreement with Ilg and Wagner's choice, a snow load of 18 pounds per square foot of horizontal projection will be used.

Art. 6. Wind Load.

There is even more difference of opinion and uncertainity as to the actual wind load to be used, than there is to the snow load. There are several difficulties encountered in attempting to properly investigate this subject, of which some are as follows:

1. A purely mathematical discussion does not give the actual pressure, for there are several conditions which cannot be stated in mathematical terms, such as the cushioning effect of the air in front of the surface, and the tendency of the air to form a vacuum behind it;

2. On account of the above it has been necessary to result to experiments in order to form any conclusions as to the true pressure. These experiments have nearly all been more or less crudely performed, and there has been considerable inability to produ ce actual conditions during the tests;

3. The roof will not, of course, carry the whole force of the wind acting horizontally, nor is it absolutely correct to use the normal component of the wind (though this is always done), for this component is slightly less than the actual normal force of the wind; and

4. This case is not analogous to a jet of water striking an inclined surface, "for water escapes laterally against a comparatively unresisting medium, while the wind particles deflected by the roof are turned off into a stream of similar air also in motion, which affects their lateral progress".

By the principles of mechanics the wind pressure P on a plane surface, in pounds per square foot, normal to the directionof the wind, has been shown to vary as the square of the velocity V in miles per hour. Some of the formulas derived from experiments are:

$P = 0.00492\,V^2$ by John Smeaton, in 1759.
$= 0.00492\,V^2$ by Colonel Duchemin, in 1842.
$= .0073 \quad .0034\,V^2$ by the French officers, Piobert, Marin, and Didion in 1838.
$= .00306(1 \quad .0486\,C)\,V^2$ by Hayer, where C is the perimeter of the surface.
$= .00340\,V^2$ by H. Allen Hazen in 1886.
$= .00350\,V^2$ by W. H. Dines in 1889.
$= .00400\,V^2$ by C. F. Marvin in 1890 at Mt. Washington.
$= .00390\,V^2$ by S. P. Langley in 1890.
$= .00360\,V^2$ by T. E. Staunton in 1903.
$= .00492\,V^2$ by Rause.
$= .00360\,V^2$ by Froude.
$= .00250\,V^2$ by Nipher.
$= .00400\,V^2$ by Hutton.

Nipher's formula gives much lower results than the older formulas. His experiments (made since 1903) were conducted most carefully; and German experiments show that the normal wind pressure has been largely overestimated in the past. This is due to the fact that the effect of the currents of wind through openings in the building, which tend to counteract the stresses in the roof, have not been fully taken into account.

To show the difference between Nipher's formula and the

older ones of which Hutton's is representative, the following table from Ilg and Wagner's thesis is presented.

Table 1.

Comparative Wind Pressures.

Wind Velocity Miles per hour.	Nipher $P = .0025\ V^2$ Pressure in pounds per square foot.	Hutton $P = .004\ V^2$ Pressure in pounds per square foot.
10	0.25	0.40
20	1.00	1.60
30	2.25	3.60
40	4.00	6.40
50	6.25	10.00
60	9.00	14.40
70	12.25	19.60
80	16.00	25.60
90	20.25	32.40
100	25.00	40.00

For the proper interpretation of the velocities given above, the following table is given, from Ketchum's "Steel Mill Buildings".

Table II.

Defined Velocities.

Wind Velocity Miles per hour.	Description.
10	Fresh breeze.
20	
30	Strong Wind.
40	High Wind.
50	Storm.
60	Violent Storm.
70	
80	Hurricane.
90	
100	Violent Hurricane.

The Chicago, New York,and Boston building laws, also the Baltimore and Ohio railroad, specify that a pressure on a vertical surface of wo pounds per square foot be taken; and from the fact that this value would be equivalent to a wind velocity of nearly 90 miles per hour according to Hutton and about 110 miles per hour according to Nipher, it is seen that a velocity in excess of these

rarely occurs. In case such a velocity is liable to occur larger values are but seldom taken, reliance being placed on the factor of safety.

"The velocity of the wind during the St. Louis tornado was 120 miles per hour, but the records of the United States Weather Bureau for the last en years show only one instance where the velocity of the wind as indicated by the anemometer, was more than 90 miles per hour".

In a high building due account must be taken of the increase in pressure at a high elevation above the earth. The variation in velocity, and consequently in pressure, due to increasing height is shown by the following table taken from C. E. Noerenberg's thesis, and compiled from Stevenson's experiments.

Table III.

Variation of Pressure with Altitude.

Feet above ground	5	9	15	25	52
Velocity in miles per hour.	4	6	6	7	8
	7	17	18	21	23
	13	23	25	30	32
	19	28	31	35	40
	26	32	34	37	43

From the above data it would seem that a horizontal pressure on a vertical surface of 30 pounds per square foot would usally be ample, and would, in accordance with such specifications as quoted above, be used by the author; but as Ilg and Wagner chose a force of 25 pounds, it will be necessary to use this same value in this thesis.

By the ordinary resolution of forces Ilg and Wagner found the normal component of this 25 pounds to be 11 pounds per square foot of horizontal projection: since it is improbable that a maxi-

mum snow and wind load would ever occur together and since in combining all the loads to figure the total load on the roof, they used the maximum ice and snow value, therefore they reduced the maximum probable wind in combination with this from 11 to 9 pounds per square foot of horizontal projection. This value will be used here.

Art. 7. Total Roof Loads.

In summing up the loads for which a truss is to be designed the weight of the roof covering, purlins, snow, and the truss itself, plus the wind pressure, must all be taken into account.

For an accurate design, each factor would have to be taken in its true value for that particular locality and truss. However, in practice it is quite customary to assume some loading expressed in pounds per square foot horizontal projection, which value is known to be large enough to give a satisfactory design. But for a large truss and when absolute accuracy is desired, stresses should be figured separately for the above factors, and the resulting maximum stress be taken.

In this thesis, a loading of 40 pounds per square foot of horizontal projection is taken; this being good practice in this locality for this type and size of truss, besides being the value used by Ilg and Wagner, and of course, this must be used here.

That this load is large enough is shown by the following data:

Weight of Truss	5.10	pounds per square foot.
Weight of Snow	18.00	horizontal projection.
Wind	9.00	
Roof Covering	3.30	
Purlins	3.00	
	38.40	

The truss weight used was approximated from Ketchum's formula for span of 100 feet, 16 feet center to center, this being the largest truss used in this work.. It is necessary to assume a truss weight from some such formula, for of course the true weight cannot be known till after design. The most common formulas for truss weights are,

Author	Total Wt. in lbs.	Wt.-Sq.ft.hor.proj.
Merriman	$3/4\ aL\ (1 + L/10)$	$3/4\ (1 + L/10)$
Maurer	$aL\ (1 + L/25)$	$(1 + L/25)$
Ricker	$aL^2(1/25 + L/6000)$	$L\ (240 + L/6000)$
Ketchum	$PaL/45(1 + L/5\sqrt{a})$	$0.89\ (1 + L/5\sqrt{a})$
Fowler	$aL\ (0.05\ L + 0.5)$	$(.05L + .5)$

Where

P = Load capacity in pounds per square foot, horizontal projection.

a = distance - center - center of trusses.

L = span in feet.

These formulas are plotted on page 43, the graph being taken from Ilg and Wagner's thesis.

Art. 8. Design of Trusses.

The compression formula of 16,000 - 70(L/r) is now quite generally used throughout the country, and its use is rapidly growing. It was the author's intention to use this formula, since it was also used by Ilg and Wagner; but tables of design of angles, found in a book entitled "Roof Trusses" by H. C. Hearne, of New York, - McGraw Hill Company, were used, which greatly shortened the otherwise lengthy work. The compression formula of 15,200 - 80 (L/r) was used, being the one nearest to the above named formu-

la to be found in this book. The results of the two differ only slightly for the values of L/r used in this investigation.

In the design, the full section was taken as being effective, 16,000 pounds per square inch being used in tension, with one 7/8 inch hole taken out.

The style of truss used together with stresses and design are tabulated on pages 30 to 33 inclusive. The type was governed, as before noted, by the maximum allowable span of corrugated steel roofing, i. e. 5.87 feet. The trusses are considered as resting on side walls, and are not fastened to columns or knee braces.

III. ECONOMIC STUDY.

Art. 9. Economy of Types.

Of the Fink, Howe, and Pratt trusses, the first is accepted as being the most economical, chiefly caused by the simple details, duplicate sections, and short compression members. In this work the lower chord is designed as horizontal, though ofteh it is necessary for clearance, or desirable for prevention of the optical illusion of sag, to give it a slight camber. This is undesirable in that even a small camber increases the weight considerably. The following table showing the economical type is quoted from Ilg and Wagner's thesis.

Table IV. Economic Type.

Type	span	Rise	Pitch	Spacing	Weight	Details	Relative Efficiency.
Pratt	60	15	1/4	16	2815	20%	89.1%
Howe	60	15	1/4	16	2816	21%	89.0%
Fink	60	15	1/4	16	2517	18%	100.0%

The Fink type was studied with regard to its economy with varying span, rise, and spacing.

Art. 10. Economic Pitch.

Thirty three trusses were investigated, the stresses being determined graphically, and indicated in tables 7 to 10, pages 30 to 33 inclusive. The design was made by the use of the book mentioned in Article 8, and a careful effort was made to pick out the lightest sections. The data for twenty-seven of these trusses is given in Table V below, and the remaining six are tabulated in the next article.

Table V. Economic Pitch.

Span	Pitch	Spacing	Weight Lbs.	Hor. Sq. Ft.
40	23°	16'	1148	1.80
40	24°	16'	1164	1.82
40	25°30'	16	1186	1.85
40	26°	16	1204	1.89
40	27°	16	1225	1.92
40	28°	16	1244	1.95
40	32°	16	1401	2.19
60	24°	16	2586	2.70
60	27°	16	2428	2.53
60	28°	16	2440	2.54
60	29°	16	2419	2.52
60	31°	16	2449	2.56
60	32°	16	2495	2.60
80	24°	16	4182	3.26
80	27°	16	4193	3.27
80	28°	16	4184	3.26
80	29°	16	4235	3.31
80	31°	16	4446	3.47
80	32°	16'	4408	3.45
100	24°	16'	6675	4.17
100	27°	16	6765	4.23
100	28°	16	6429	4.01
100	28°30'	16	6500	4.06
100	29°	16	6571	4.10
100	29°30'	16	6658	4.17
100	31°	16	6846	4.27
100	32°30'	16	7503	4.70

The pitches shown were chosen so that they might give points in between those found by Ilg and Wagner. The results together with their curves are shown graphically on pages 34 to 37 inclusive, and a discussion of these curves is given in Article 16.

Art. 11. Economical Spacing.

It was proposed to show here the variation of purlin with the variation in distance center - center of trusses, in a little more detail than given by Ilg and Wagner, and also the variation of combined weight of truss and purlin with the varying spacing. For that purpose, six trusses as shown in Table VI were chosen,

and the data computed. The particular points chosen were taken because they fitted in between those given in the previous thesis mentioned.

It was thought that the points which would be found would lie somewhere in the general neighborhood of those given by Ilg and Wagner, and that the variation of weights would then be shown a little more in detail. But it is to be noted that the results of the two investigations do not agree very closely, particularly the weights, in fact nearly all being less than those found by Ilg and Wagner.

It is therefore quite evident that the two investigations were conducted along somewhat different lines, for it is obvious that no error in computations could be carried consistently throughout, and that the two sets of data were not computed on the same basis.

For example, Messrs. Ilg and Wagner failed to show sketches of the type used, so that the exact point of transition from straight Fink to Fan, and back to Fink, is not known. Though they state that the plan used by them (the same as was used in this thesis), was not to exceed the panel length governed by the maximum allowable span of corrugated steel, i. e. 5.87 feet, they may have allowed some leeway on each side of this, and made their change of type at different points from those used here. In this work but very little leeway was allowed over this dimension, not more than about 0.05 foot. The type used for each span is given in Tables 7 to 10, pages 30 to 33 inclusive.

Furthermore, the author of this thesis made a careful effort to use the most economical sections throughout, and all compu-

tations were checked. Therefore, the work is thought to be reliable. One can but naturally conclude that the possible difference in transition of types used, with other somewhat small factors of design, concerning which they did not give detailed information, and wherein we may have differed, though affecting the weights to some degree, could not have caused such a difference as exists. This difference, then, can be accounted for only through their possible failure to secure this most economic section. It is to be noted that design with them was a more laborious process, involving greater possibilities for error than the author encountered, since they had to use the usual methods of design, while the author had access to the work previously noted, which simplified design greatly and gave at a glance the most economic section.

Their curves for variation of weight with variable spacing of trusses, together with those plotted from the following data, are shown on pages 40 and 41.

The data in this connection, from the author's study is shown in Table VI. The trusses cost about 4 cents a pound, and the purlins about 2 cents. Therefore the combined cost would be given by $4T + 2P$, T being truss weight, and P purlin weight in pounds per horizontal square foot. This relation, showing relative costs of metal for different spans is shown among the curves on page 42.

Table VI.

Economic Spacing of Trusses.

Span	Pitch	Spacing	Truss Weight Lbs.Hor.Sq. Ft.	Purlins - Z-bars Lbs. Hor. Sq.Ft.	Combined Truss & Purlins	4T + 2P
60	1/4	14	2.75	2.23	4.98	15.46
60	1/4	18	2.42	2.50	4.92	14.68
80	1/4	14	3.49	2.18	5.67	18.32
80	1/4	18	3.04	2.46	5.50	17.08
100	1/4	14	4.55	2.16	6.71	22.52
100	1/4	18	3.93	3.44	6.37	20.60

The economic spacing would, of course, be governed by the combined weight of truss and purlin, and so purlins were designed for the series of trusses listed above.

Here again information was lacking, in that Messrs. Ilg and Wagner did not state what shape they used for purlins. The Z-bar is most economical so far as strength for a given weight is concerned. For purlins Z-bars were designed, and values plotted with those of Ilg and Wagner, on page 41. It is quite evident from these curves that they did not use Z bars. While it would be a simple matter to discover what shape they used, this has not been done. The reason follows: they drew curves showing variation of combined weight of truss and purlin with varying span, and as before mentioned, since it was thought that the points to be found would lie rather closely to their curves, only two spacings of trusses were taken, 14 and 18 feet. However their truss weights and those found here differ so much that even should the shape which they used for purlins be added to the author's truss weights the resulting points would differ so much from theirs that no conclusion could be drawn.

Notwithstanding this, the work indicated above would have been performed and shown graphically, except for the unfortunate

fact that, for reasons stated before, weights for but two spacings of 1/4 pitch were figured for any one span, and these, giving but two points could (for lack of more information) result in but a straight line. It is evident that the graphs for economic spacing would be curves, probably of the same general nature as that shown by Ilg and Wagner. So nothing could be learned from a plot of their curves and the points found here, and they have not been drawn. It is to be regretted that more information on this subject is not available, but the possibility of the weights differing so much was quite unlooked for, and hence the points chosen so few.

Art. 12. Effect of Span on Weight.

The effect of variation of span in trusses of constant spacing is shown in Tables V and XI, and is shown graphically on pages 34 to 40, and 43. In the computation of weights, the details were taken as 20 per cent of the bare weight of truss.

IV. WEIGHT OF STEEL TRUSSED ROOFS.

Art. 13. Computation of Weights.

Messrs. Ilg and Wagner computed the weights of their trusses of 40, 60, 80, and 100-foot spans, as given by the common formula, and tabulated them. As none of the formulas expresses the weight in terms of the pitch, all pitches of one span figured from these formulae, give the same weight. Table XI has been taken from their thesis, and shows in addition how their actual truss weights in pounds per square foot of horizontal projection compare with those computed here, and with the common formulae.

Art. 14. Weight Formula.

Curves were plotted for pitches of 1/3, 1/4, 1/5, and 30°, with spans as abscissae and weights as ordinates. These are shown on page 38. The average curve was drawn and its equation derived, taking the general formula $Y = a + bx + cx^2 + dx^3$. With values of Y from the spans of 40, 60, 80, and 100 feet, the constants were determined and the resulting equation was:

$$W = -1082 + 76.5L - 0.751L^2 + 0.00812L^3.$$

This is for the total weight of a truss of span L, spaced 16 feet - 0 inches center - center. To reduce to pounds per horizontal square foot, dividing by 16 x L gives:

$$W = -68/L + 4.8 - 0.047L + 0.000505L^2.$$

This is for a loading of 40 pounds per horizontal square foot.

Of the common formulas for truss weights, none is a function of the pitch, while for those trusses used here, the weight varies with a maximum value of about 12 per cent from the average

curve. An attempt was made to derive a formula which should be a function of both the pitch and span. A mathematical solution was reached, but not a practical one, for the formula thus derived does not give as good results as the equation of the average curve. In fact, a glance at the curves on page 38 will show that no formula could be gotten, at least so far as this data is concerned, which would be a function of the pitch. For it is clearly seen that there is no general law of variation, and the points do not lie in any particular order. How well the average curve fits the pitches shown is best seen from the curves. As before stated the maximum variation from the average is about 12 per cent.

This variation from the average is shown in curves on page 39. It is noticed that the shorter the span, the more uniform is the variation, and the less the departure from the average.

V. CONCLUSIONS.

Art. 15. Economical Type.

It is shown by Table IV page 18 that for trusses of this size, the Fink type is the most economical.

Art. 16 Economical Pitch.

The economical pitch may be picked out from the curves on pages 34 to 37inclusive.

The 40-foot span has an economic pitch of probably 20°, though as shown by the curves, this is rather indefinite. For the other three spans, a pitch of 28° - 29° seems the most economical. It is seen that 1/4 and 30° pitches do not give results varying greatly except for 100-foot span, where 30° is the more economical.

Though these curves show that for the three higher spans the law of variation is about the same as shown by Ilg and Wagner, the points found do not lie along a smooth curve; and in the case of the 40-foot span the exception to their law of variation is quite marked.

The author believes that their point for 40-foot span, 1/5 pitch, is in error, for in the trusses of shorter span the governing factor of design is usually not the stress, but the minimum allowable size of angles. That is, in nearly all members, the sizes are the same, and hence as the pitch decreases, the lengths of members decreases, and consequently the weight would be falling off in a regular ratio, and through a much greater range of pitches than the other spans where the stresses, and L/r govern. Of course, when the pitch

decreases considerably below 20°, the chord stresses will rise greatly and will govern the design, causing the curve to rise again. But it will be clearly seen that in this case the curve would be much flatter and the weights would vary in a much smaller ratio with the pitch. The author's curve for 40-foot span being determined by six points beyond that point when the curvature differs from Ilg and Wagner's is by the law of mathematical probability more nearly correct than their curve which has but one point from the 1/4 to the 1/5 pitch.

In the spans of 60, 80, and 100 feet, the points do not lie along a smooth curve. This is but natural as there are so many different factors of design, such as minimum section, L/r, and stress in the member. While each one separately varies uniformly with the pitch, taken together the law of variation, and the resulting variation in the weight of the truss becomes very complicated. Then again, the inability to pick commercial sizes of angles which vary in direct proportion to the stress, further complicates matters. Though the most economical section be taken, it is but seldom that there is not some excess area over that which is required.

Thus it is seen that the points could not lie along a smooth curve except, as noted, in the 40-foot span.

Art. 17. Economical Spacing of Trusses.

It is unfortunate that this subject cannot be treated as thoroughly as was intended at the outset of this study, due to the unlooked for scarcity of data as explained in Article 11. This same article however, gives data concerning the approximate relative cost of combined truss and purlin material with varying spans, and spacing

of trusses, as plotted on page 42.

From these curves we conclude that though the exact law cannot be given on account of lack of data, still it is evident that up to some certain limit the cost decreases as the spacing of trusses increases, and that for a given spacing, the cost varies in almost a uniform ratio with the span.

Art. 18. Effect of Span on Weight.

The effect of span is shown on page 38 and also on page 43. In this latter set of curves, the graphs of the common truss weight formulae have been plotted as shown by Ilg and Wagner, and also the author's equation of the average curve in pounds per horizontal square foot. Naturally enough, the truss weight increases with the span.

Art. 19. Weights of Steel Trusses Roofs.

The summary of Results, Table XI, page 44, gives a comparison of common truss formulas, in addition to Ilg and Wagner's formula, the author's design, and his equation of the average curve.

The common formulas appear somewhat too heavy, particularly that of Merriman, but this is explained by Ilg and Wagner, who found the same discrepancy between the common formulae and their own formula, by the fact that the formulas given were deduced when the unit stresses used were lower than today, and that the formulas may have been deduced from roofs designed for heavier loading.

The formula for 16 feet center - center, and loading of 40 pounds per square foot horizontal projection, as derived from the average curve, has been discussed in Article 14, and its limitations

given there, are best shown by the curves.

It is evident that a new general formula is in demand for weights of roof trusses, which will give lighter values than any of the present ones; but it is also quite obvious that, however desirable, it is impossible to deduce a formula in terms of both the pitch and span.

Span 40' c-c 16'

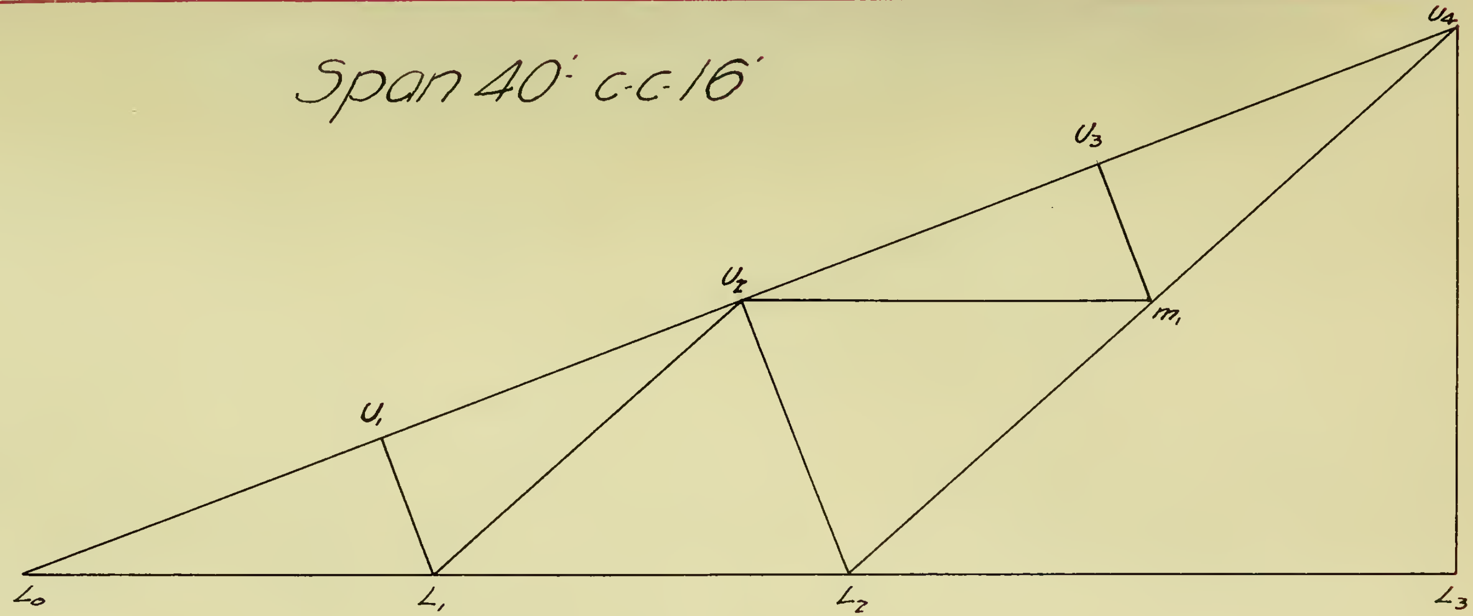


Table VII

Pitch	Member	L_0U_1	U_1U_2	U_2U_3	U_3U_4	L_0L_1	L_1L_2	L_2L_3	U_4L_3	L_2m_1	m_1U_4	U_2L_2	L_1U_2	U_2m_1	U_1L_1	U_3m_1
23°	Stress	-28.5	-27.3	-26.0	-24.8	+26.3	+22.4	+14.8	All stresses = 0	+7.6	+11.4	-5.9	+3.8		-3.0	
	Section	2-3"×2½"×¼"				2-2½"×2½"×¼"		1-2½"×2½"×¼"	1-2½"×2½"×¼" used throughout	1-2½"×2½"×¼"						
24°	Stress	-27.6	-26.3	-25.0	-23.7	+25.2	+21.7	+14.5		+7.2	+10.8	-5.9	+3.6		-2.9	
	Section	2-3"×2½"×¼"				2-2½"×2½"×¼"		1-2½"×2½"×¼"		1-2½"×2½"×¼"						
25°30'	Stress	-26.1	-24.7	-23.3	-22.0	+23.5	+20.1	+13.4		+6.8	+10.2	-5.8	+3.4		-2.9	
	Section	2-3"×2½"×¼"				2-2½"×2½"×¼"		1-2½"×2½"×¼"		1-2½"×2½"×¼"		1-3"×2½"×¼"	1-2½"×2½"×¼"			
26°	Stress	-25.5	-24.1	-22.7	-21.3	+22.9	+19.6	+13.0		+6.6	+9.9	-5.8	+3.3		-2.9	
	Section	2-3"×2½"×¼"				2-2½"×2½"×¼"		1-2½"×2½"×¼"		1-2½"×2½"×¼"		1-3"×2½"×¼"	1-2½"×2½"×¼"			
27°	Stress	-24.7	-23.3	-21.8	-20.4	+22.0	+18.9	+12.5		+6.3	+9.5	-5.7	+3.2		-2.8	
	Section	2-3"×2½"×¼"				2-2½"×2½"×¼"		1-2½"×2½"×¼"		1-2½"×2½"×¼"		1-3"×2½"×¼"	1-2½"×2½"×¼"			
28°	Stress	-23.9	-22.4	-20.9	-19.4	+24.1	+18.1	+12.1		+6.0	+9.0	-5.7	+3.1		-2.8	
	Section	2-3"×2½"×¼"				2-2½"×2½"×¼"		1-2½"×2½"×¼"		1-2½"×2½"×¼"		1-3"×2½"×¼"	1-2½"×2½"×¼"			
32°	Stress	-21.2	-19.5	-17.8	-16.1	+18.0	+15.4	+10.3		+5.2	+7.7	-5.4	+2.6		-2.7	
	Section	2-3"×2½"×¼"				2-2½"×2½"×¼"		1-2½"×2½"×¼"		1-2½"×2½"×¼"		1-4"×4"×5/16"	1-2½"×2½"×¼"			

Span 60'· c-c-16'

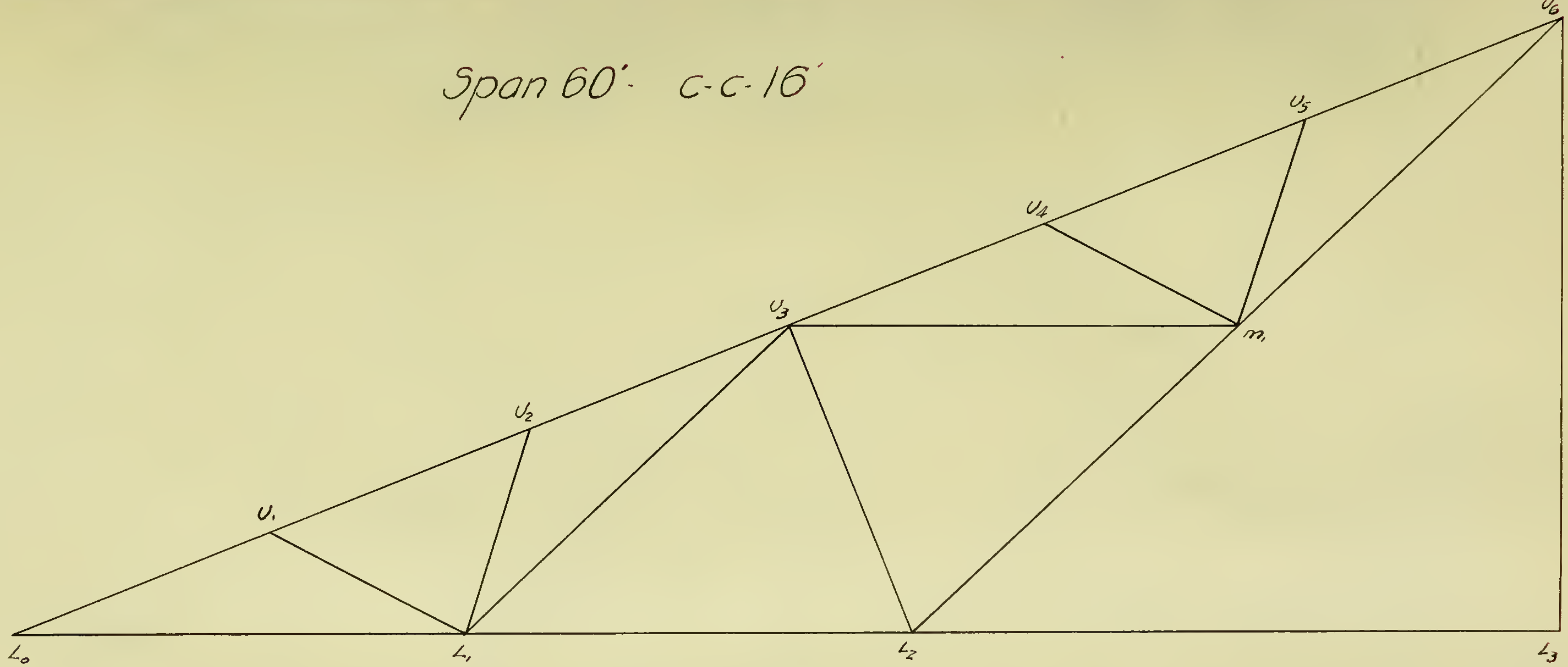


Table VIII

Pitch	Member	L_0U_1	U_1U_2	U_2U_3	U_3U_4	U_4U_5	U_5U_6	L_0L_1	L_1L_2	L_2L_3	L_3U_6	L_2m_1	m_1U_6	U_3L_2	L_1U_3	U_3m_1	U_1L_1	U_2L_1	U_4m_1	U_5m_1
24°	Stress	-46.0	-42.5	-43.7	-42.3	-38.8	-40.0	+42.6	+34.8	+23.0	All stresses=0 ∴ 1-2½"×2½"×¼" used	+11.7	+19.6	-8.9	← +7.8	→	←	-3.8		→
	Section	←	2-4"×3"×5/16"				→	2-2½"×2½"×3/8"		1-3×3×5/16		1-2½"×2½"×3/8"		2-2½×2½×1/4	←	1-2½"×2½"×1/4"				→
27°	Stress	-38.6	-35.3	-35.7	-34.3	-31.0	-31.4	+34.4	+28.1	+18.7		+9.4	+15.7	-8.6	← +6.3	→	←	-3.4		→
	Section	←	2-3½"×2½"×5/16"				→	2-3"×3"×1/4"		1-3×3×1/4		1-3"×3"×1/4"		2-3×2½×1/4	←	1-2½"×2½"×1/4"				→
28°	Stress	-37.4	-34.2	-34.4	-32.9	-29.6	-29.8	+33.0	+26.9	+17.9		+9.0	+15.0	-8.5	← +6.0	→	←	-3.3		→
	Section	←	2-3½"×2½"×5/16"				→	2-3"×3"×1/4"		1-3×3×1/4		1-2½"×2½"×1/4"		2-3×2½×1/4	1-2½"×2½"×1/4"		1-3"×2½"×1/4"			→
29°	Stress	-36.3	-33.0	-33.2	-31.7	-28.4	-28.6	+31.8	+25.9	+17.2		+8.7	+14.5	-8.4	← +5.8	→	←	-3.3		→
	Section	←	2-3"×2½"×5/16"				→	2-3"×3"×1/4"		1-3×3×1/4		1-2½"×2½"×1/4"		2-3×2½×1/4	1-2½"×2½"×1/4"		1-3"×2½"×1/4"			→
31°	Stress	-34.2	-31.1	-31.0	-29.3	-26.2	-26.1	+29.3	+24.0	+15.9		+8.1	+13.4	-8.2	← +5.4	→	←	-3.2		→
	Section	←	2-3"×2½"×5/16"				→	2-2½"×2½"×1/4"		1-3×3×1/4		1-2½"×2½"×1/4"		2-3½×2½×1/4	1-2½"×2½"×1/4"		1-3"×2½"×1/4"			→
32°	Stress	-33.0	-29.8	-29.6	-27.9	-24.8	-24.6	+28.0	+22.9	+15.2		+7.7	+17.8	-8.2	← +5.1	→	←	-3.1		→
	Section	←	2-3½"×2½"×1/4"				→	2-2½"×2½"×1/4"		1-2½×2½×1/4		1-2½"×2½"×1/4"		2-3½×2½×1/4	1-2½"×2½"×1/4"		1-3½"×3"×5/16"			→

Span-80'   c-c-16'

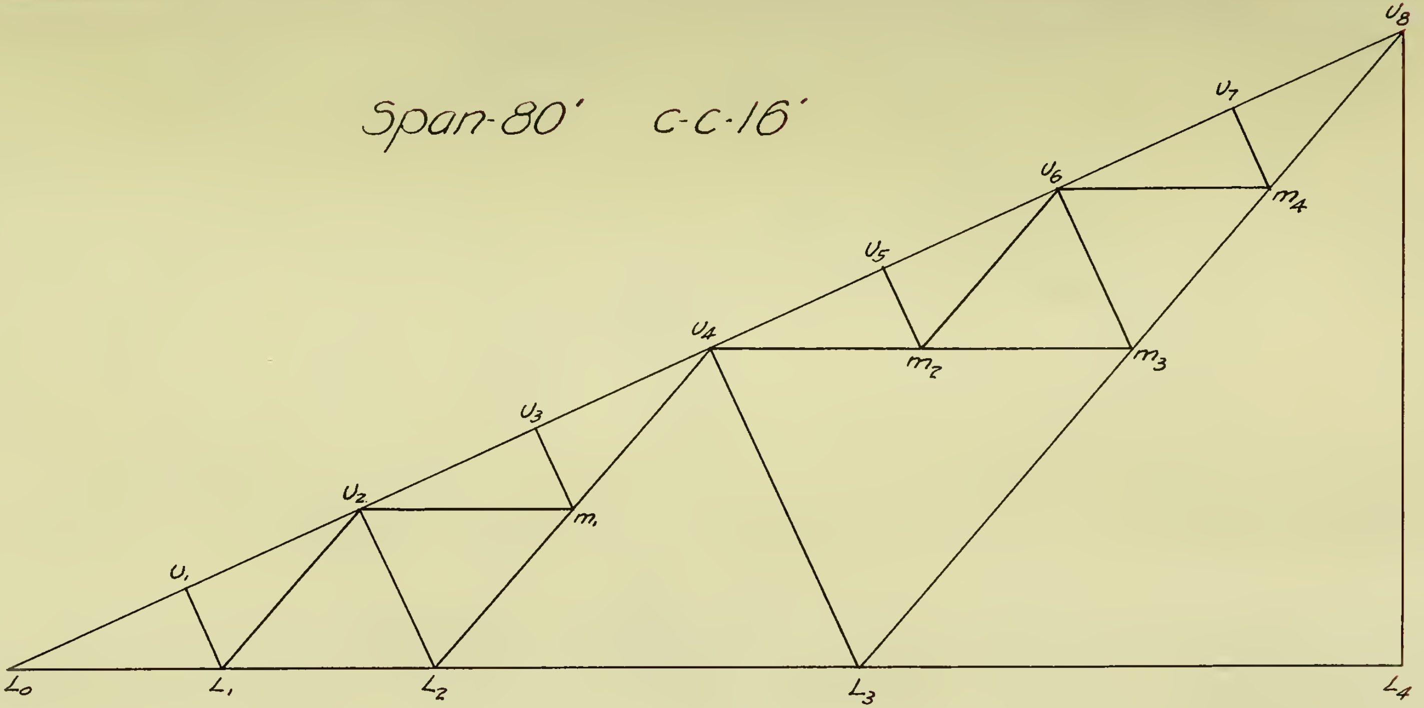


Table IX

Pitch	Member	L_0U_1	U_1U_2	U_2U_3	U_3U_4	U_4U_5	U_5U_6	U_6U_7	U_7U_8	L_0L_1	L_1L_2	L_2L_3	L_3L_4	L_3m_3	m_3m_4	m_4U_8	U_4L_3
24°	Stress	-59.2	-57.9	-56.6	-55.3	-54.0	-52.8	-51.5	-50.2	+54.2	+50.5	+43.3	+28.9	+14.5	+21.7	+25.3	-11.7
	Section	←			2-5"×3"×5/16"				→	←	2-3"×3"×3/8"	→	1-3½×3½×5/16	← 1-2½"×2½"×7/16"		→	2-3"×2½"×¼"
27°	Stress	-53.0	-51.6	-50.2	-48.7	-47.2	-45.8	-44.2	-42.9	+47.3	+44.2	+37.9	+25.3	+12.7	+19.0	+22.1	-11.5
	Section	←			2-3½"×3"×3/8"				→	←	2-3"×3"×5/16"	→	1-2½"×2½×7/16	← 1-2½"×2½"×7/16		→	2-3½×2½×¼
28°	Stress	-51.0	-49.6	-48.1	-46.6	-45.1	-43.6	-42.1	-40.5	+45.1	+42.1	+36.0	+23.9	+12.1	+18.1	+21.2	-11.3
	Section	←			2-3½"×3"×3/8"				→	←	2-3"×3"×5/16"	→	1-3×3×5/16	← 1-2½"×2½"×3/8"		→	2-3½×2½×¼
29°	Stress	-49.5	-47.9	-46.3	-44.8	-43.2	-41.6	-41.2	-38.6	+43.3	+40.4	+34.6	+23.0	+11.5	+17.4	+20.2	-11.2
	Section	←			2-4"×3"×5/16"				→	←	2-3"×3"×5/16"	→	1-3×3×5/16	← 1-2½"×2½"×3/8"		→	2-4×3×5/16
31°	Stress	-46.9	-45.3	-44.5	-41.9	-40.2	-38.6	-37.0	-35.3	+40.2	+37.5	+32.2	+21.5	+10.8	+16.1	+18.7	-11.0
	Section	←			2-4"×3"×5/16"				→	←	2-2½"×2½"×3/8"	→	1-2½×2½×3/8	← 1-3"×3"×¼"		→	2-5×3×5/16
32°	Stress	-45.0	-43.4	-41.7	-40.0	-38.3	-36.6	-34.8	-33.2	+38.1	+35.7	+30.5	+20.3	+10.2	+15.3	+17.8	-10.8
	Section	←			2-4"×3"×5/16"				→	←	2-3"×3"×¼"	→	1-2½×2½×3/8	← 1-3"×3"×¼		→	2-5×3×5/16

Pitch	Member	U_1L_1	U_3m_1	U_5m_2	U_7m_4	L_1U_2	U_2m_1	m_2U_6	U_6m_4	U_2L_2	U_6m_3	L_2m_1	m_1U_4	U_4m_2	m_2m_3	U_8L_4
24°	Stress	←	-3.0		→	←	+3.7		→	← -5.9	→	+7.2	← +11.0	→	+7.2	All stresses = 0 ∴ 1-2½"×2½"×¼" throughout
	Section	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	1-2½"×2½"×¼"	←	1-2½"×2½"×¼"			→	
27°	Stress	←	-2.9		→	←	+3.1		→	← -5.8	→	+6.4	← +9.5	→	+6.4	
	Section	←		1-2½"×2½"×¼"					→	1-3"×2½"×¼	←	1-2½"×2½"×¼"			→	
28°	Stress	←	-2.8		→	←	+3.0		→	← -5.6	→	+6.0	← +9.0	→	+6.0	
	Section	←		1-2½"×2½"×¼"					→	1-3"×2½"×¼"	←	1-2½"×2½"×¼"			→	
29°	Stress	←	-2.8		→	←	+2.9		→	← -5.6	→	+5.8	← +8.6	→	+5.8	
	Section	←		1-2½"×2½"×¼"					→	1-3"×2½"×¼"	←	1-2½"×2½"×¼"			→	
31°	Stress	←	-2.7		→	←	+2.7		→	← -5.6	→	+5.4	← +8.0	→	+5.4	
	Section	←		1-2½"×2½"×¼"					→	1-3½"×3×5/16	←	1-2½"×2½"×¼"			→	
32°	Stress	←	-2.7		→	←	+2.5		→	← -5.4	→	+5.1	← +7.7	→	+5.1	
	Section	←		1-2½"×2½"×¼"					→	1-3½"×3"×5/16"	←	1-2½"×2⅓"×¼			→	

Span-100' c-c-16'

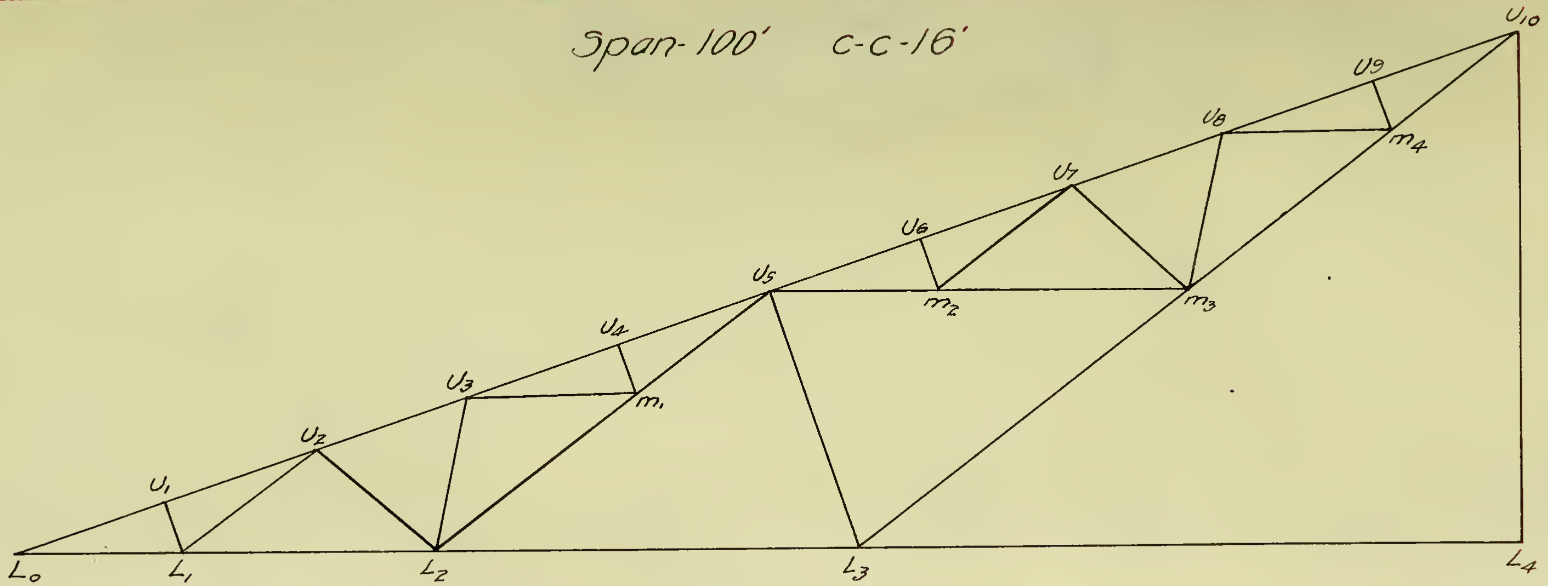
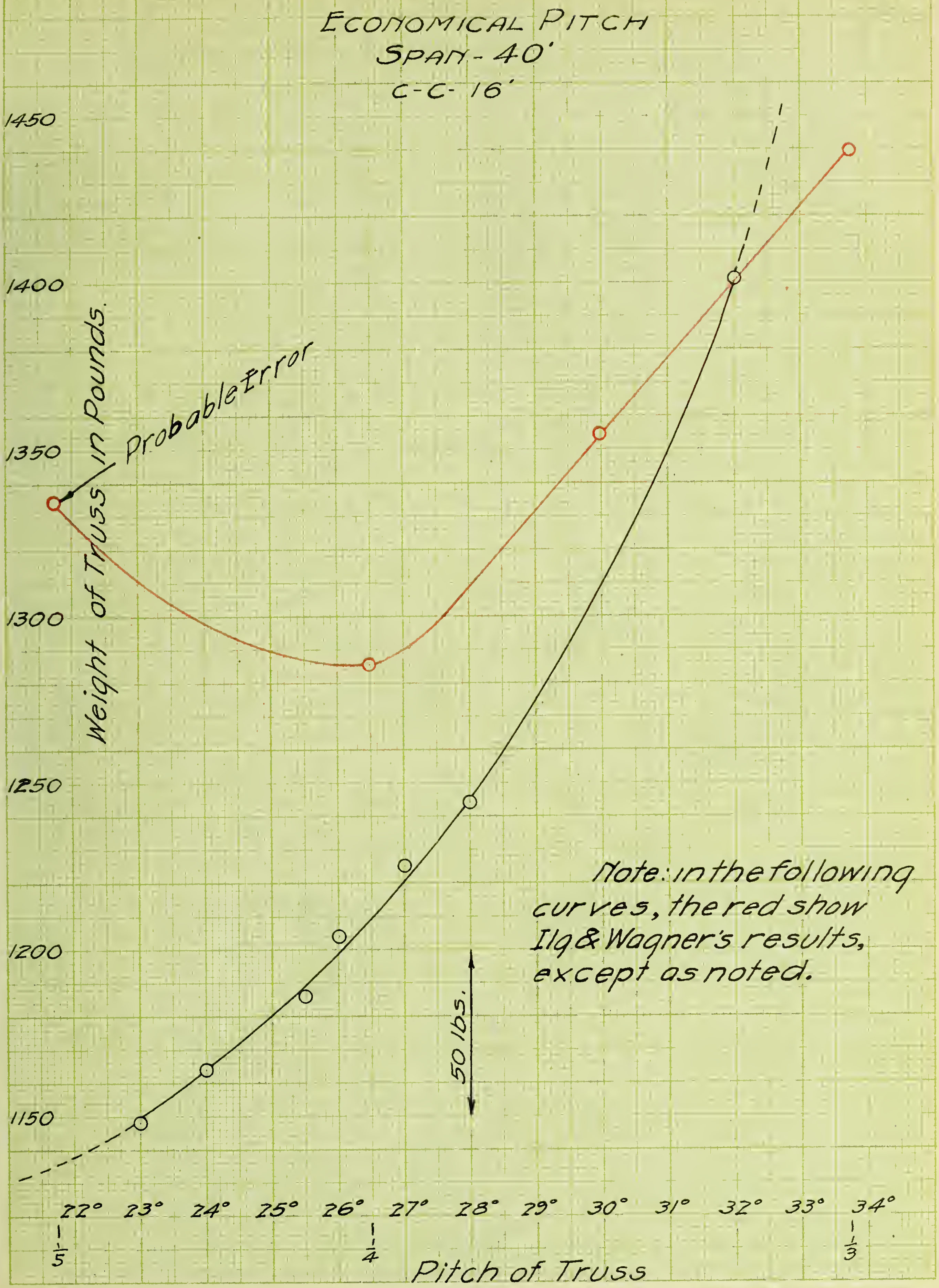


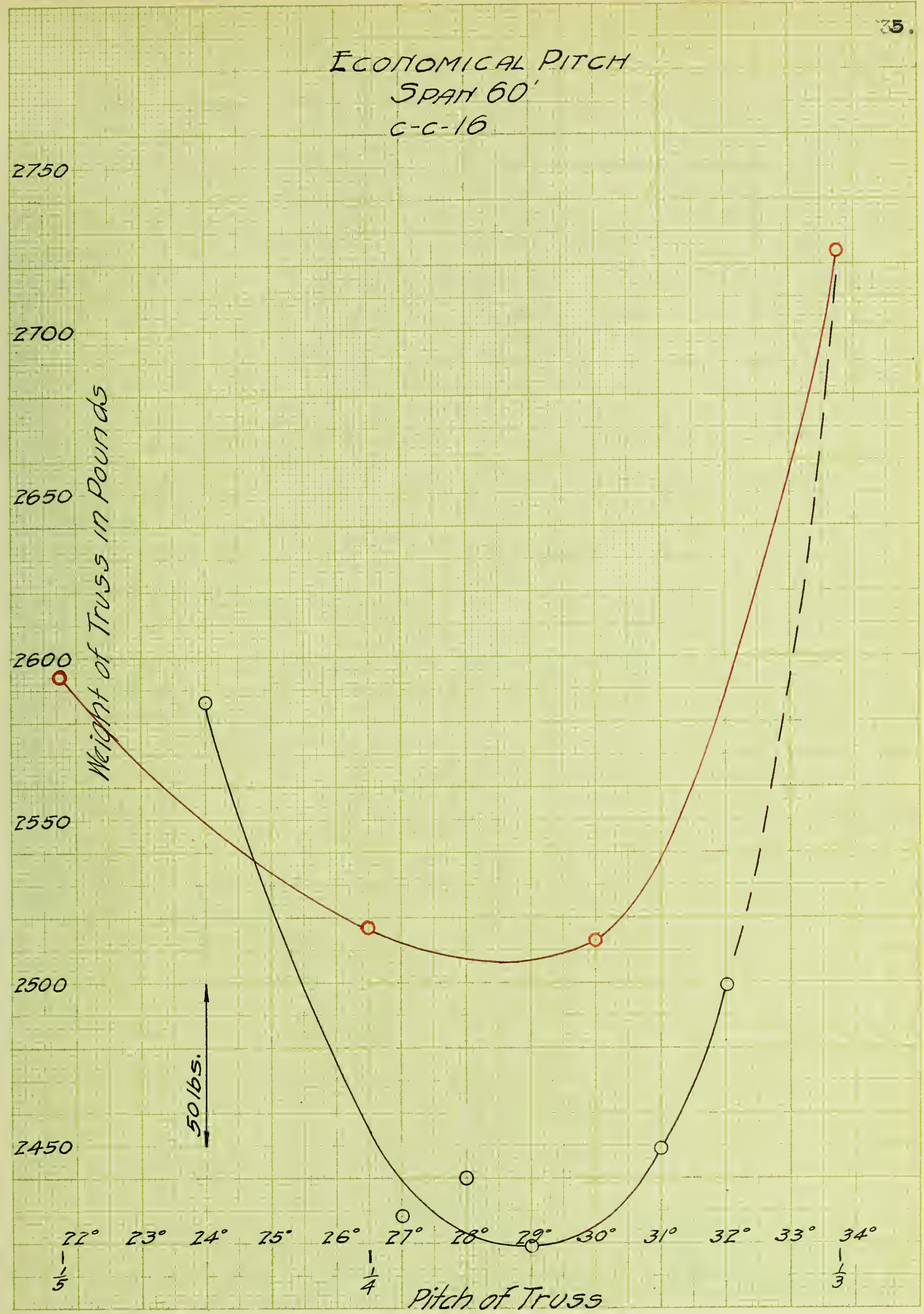
Table X

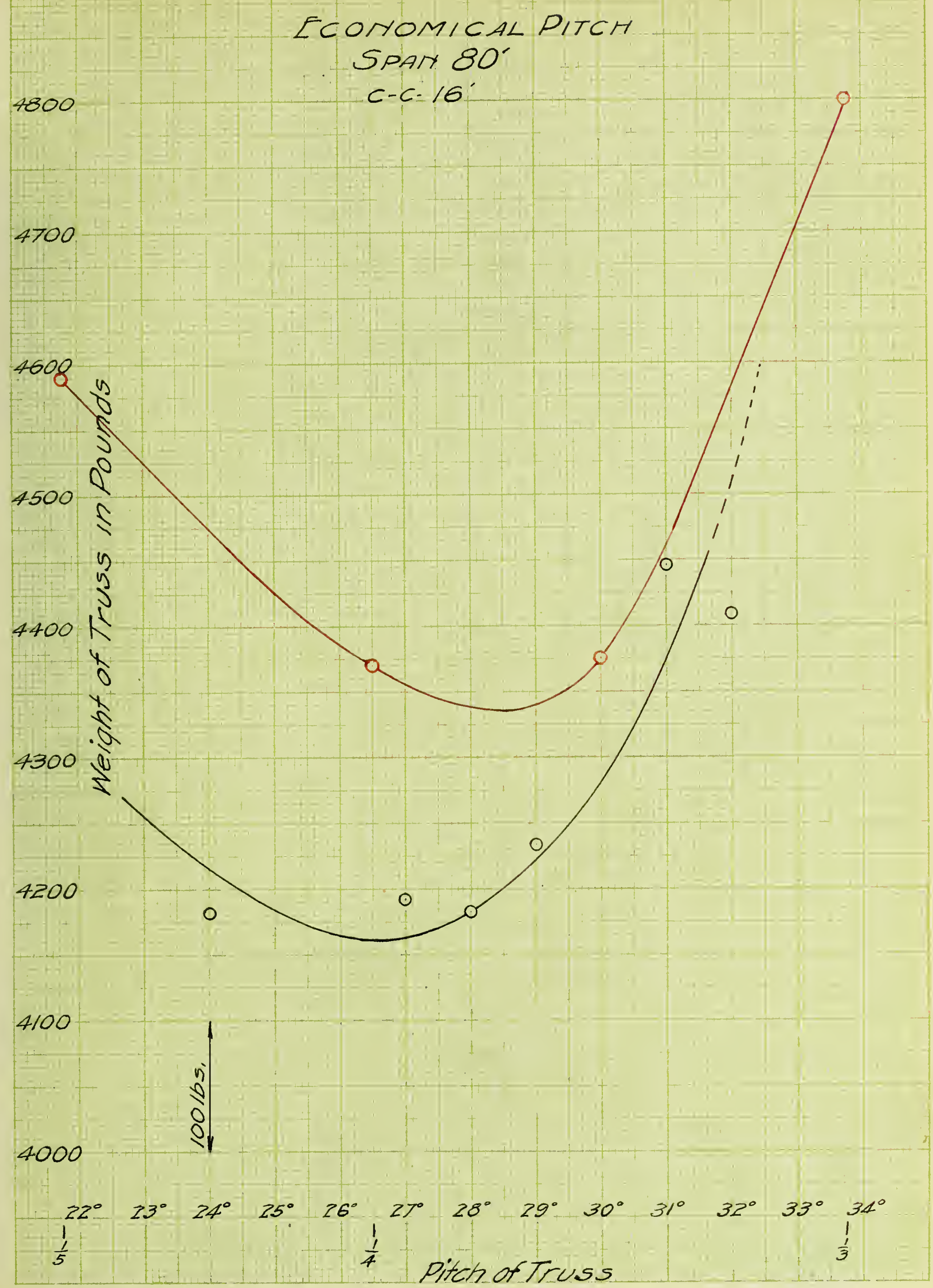
Pitch	Member	L_0U_1	U_1U_2	U_2U_3	U_3U_4	U_4U_5	U_5U_6	U_6U_7	U_7U_8	U_8U_9	U_9U_{10}	L_0L_1	L_1L_2	L_2L_3	L_3L_4
24°	Stress	-75.4	-74.0	-67.5	-71.6	-70.3	-69.0	-67.7	-61.1	-64.1	-63.9	+68.9	+64.3	+54.6	+36.4
	Section	←			2-5"×3½"×3/8"						→	2-3"×3"×½"			1-3×3×½
27°	Stress	-67.0	-65.5	-59.7	-62.7	-61.2	-59.8	-58.3	-52.4	-55.4	-54.0	+59.8	+56.7	+47.3	+31.4
	Section	←			2-5"×3"×3/8"						→	2-4"×4"×5/16"			1-4×4×5/16
28°	Stress	-64.8	-63.3	-57.5	-60.3	-58.8	-57.3	-55.8	-50.0	-52.8	-51.3	+57.3	+54.3	+45.3	+30.2
	Section	←			2-5"×3½"×5/16"						→	2-3½"×3½"×5/16"			1-4×4×5/16
28°30'	Stress	-63.9	-62.3	-56.5	-59.3	-57.7	-56.1	-54.5	-48.9	-51.5	-50.1	+56.1	+53.2	+44.3	+29.6
	Section	←			2-5"×3½"×5/16"						→	2-3½"×3½"×5/16"			1-4×4×5/16
29°	Stress	-62.9	-61.3	-55.5	-58.2	-56.6	-55.1	-53.6	-47.9	-50.5	-49.0	+55.0	+52.0	+43.2	+28.8
	Section	←			2-5"×3½"×5/16"						→	2-3½"×3½"×5/16"			1-3½×3½×5/16
29°30'	Stress	-61.9	-60.3	-54.8	-57.3	-55.7	-54.0	-52.5	-47.0	-49.3	-47.8	+54.0	+51.0	+42.5	+28.4
	Section	←			2-5"×3½"×5/16"						→	2-3½"×3½"×5/16"			1-3½×3½×5/16
31°	Stress	-59.1	-57.5	-52.0	-54.2	-52.6	-50.9	-48.3	-44.0	-46.0	-44.3	+50.7	+48.0	+40.0	+26.7
	Section	←			2-5"×3"×5/16"						→	2-2½"×2½"×7/16"			1-3¼×3¼×5/16
32°30'	Stress	-56.9	-55.2	-50.0	-51.8	-50.0	-48.3	-46.6	-41.5	-43.2	-41.5	+48.1	+45.6	+37.9	+25.4
	Section	←			2-5"×3"×5/16"						→	2-3"×3"×5/16"			1-2¾×2¾×7/16

Pitch	Member	$U_{10}L_4$	L_3m_3	m_3m_4	m_4U_{10}	U_5L_3	U_1L_1	U_4m_1	U_6m_2	U_9m_4	L_1U_2	U_3m_1	m_2U_7	U_8m_3	U_2L_2	L_2U_3	U_7m_3	m_3U_8	L_2m_1	m_1U_5	U_5m_2	m_2m_3
24°	Stress	0	+18.2	+29.1	+37.9	-14.6	←	-2.9		→	←	+3.6		→	←	-4.9		→	+10.8	← +14.5	→	+10.8
	Section	All stresses = 0 ∴ 1-2½"×2½"×¼" throughout.	1-4"×4"×5/16"			2-3½×2½×¼	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	←	2-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→
27°	Stress		+15.7	+25.2	+28.3	-14.3	←	-2.8		→	←	+3.1		→	←	-4.6		→	+9.5	← +12.5	→	+9.5
	Section		1-3½"×3½"×5/16"			2-5"×3"×5/16	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	←	1-4"×4"×5/16"		→	←	1-2½"×2½"×¼"		→
28°	Stress		+15.0	+24.1	+27.1	-14.1	←	-2.8		→	←	+3.0		→	←	-4.5		→	+9.0	← +12.0	→	+9.0
	Section		1-3"×3"×3/8"			2-5"×3"×5/16	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	←	1-4"×4"×5/16"		→	←	1-2½"×2½"×¼"		→
28°30'	Stress		+14.7	+23.7	+26.5	-14.0	←	-2.8		→	←	+3.0		→	←	-4.5		→	+8.8	← +11.8	→	+8.8
	Section		1-3½"×3½"×5/16"			2-5"×3"×5/16	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	←	1-4"×4"×5/16"		→	←	1-2½"×2½"×¼"		→
29°	Stress		+14.5	+23.3	+26.2	-13.9	←	-2.8		→	←	+3.0		→	←	-4.5		→	+8.8	← +11.8	→	+8.8
	Section		1-3½"×3½"×5/16"			2-5×3×5/16	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	←	2-3"×2½"×¼"		→	←	1-2½"×2½"×¼"		→
29°30'	Stress		+14.1	+22.7	+25.6	-13.9	←	-2.8		→	←	+2.9		→	←	-4.5		→	+8.5	← +11.4	→	+8.5
	Section		1-3"×3"×3/8"			2-5"×3"×5/16	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	←	2-3"×2½"×¼"		→	←	1-2½"×2½"×¼"		→
31°	Stress		+13.5	+21.5	+24.0	-13.7	←	-2.7		→	←	+2.7		→	←	-4.4		→	+8.1	← +10.7	→	+8.1
	Section		1-3"×3"×5/16"			2-6×3½×3/8	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	←	2-3"×2½"×¼"		→	←	1-2½"×2½"×¼"		→
32°30'	Stress		+12.6	+20.3	+22.8	-13.5	←	-2.7		→	←	+2.5		→	←	-4.3		→	+7.7	← +10.2	→	+7.7
	Section		1-3"×3"×5/16"			2-6×3½×3/8	←	1-2½"×2½"×¼"		→	←	1-2½"×2½"×¼"		→	←	2-3½"×3½"×5/16"		→	←	1-2½"×2½"×¼"		→

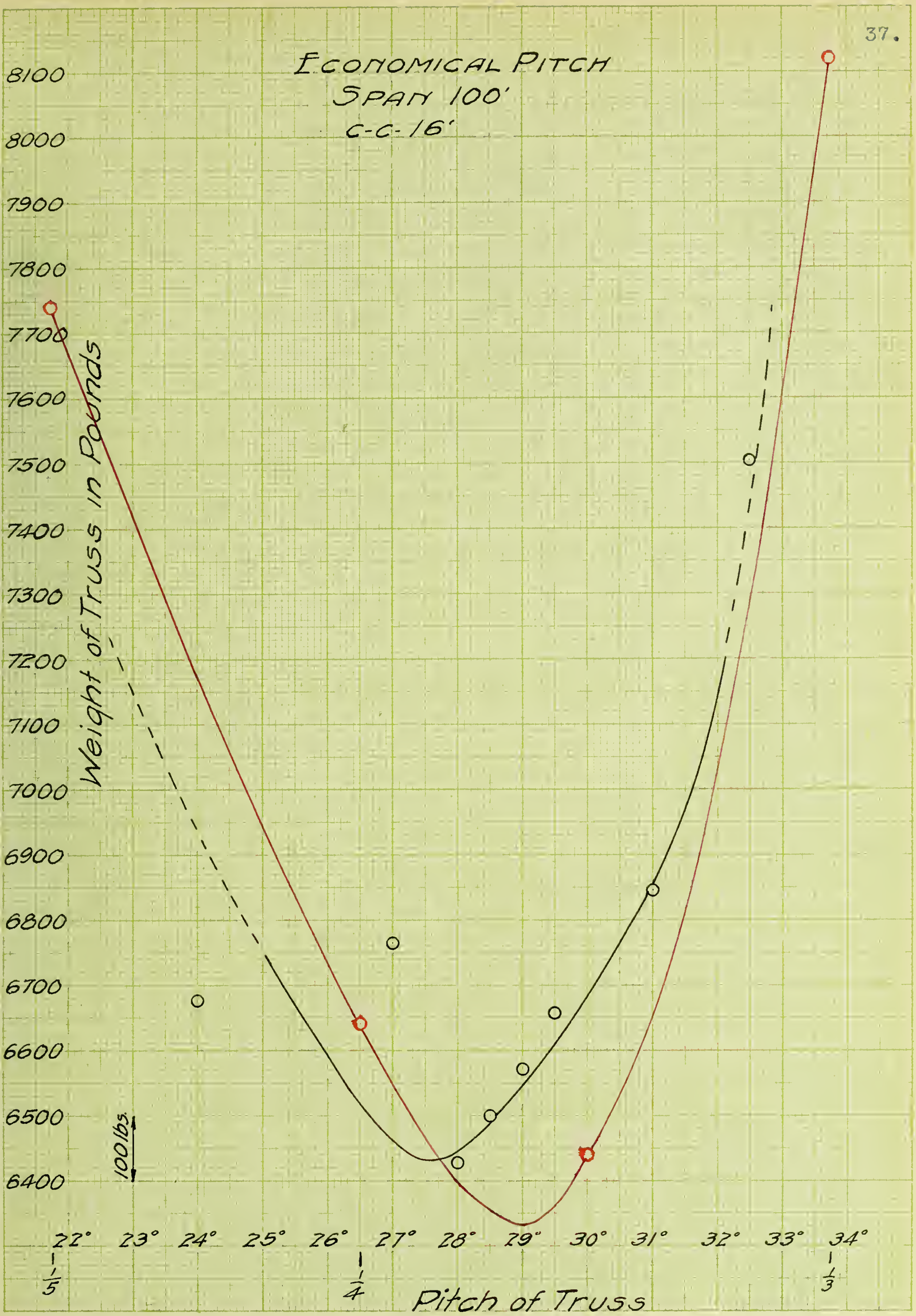


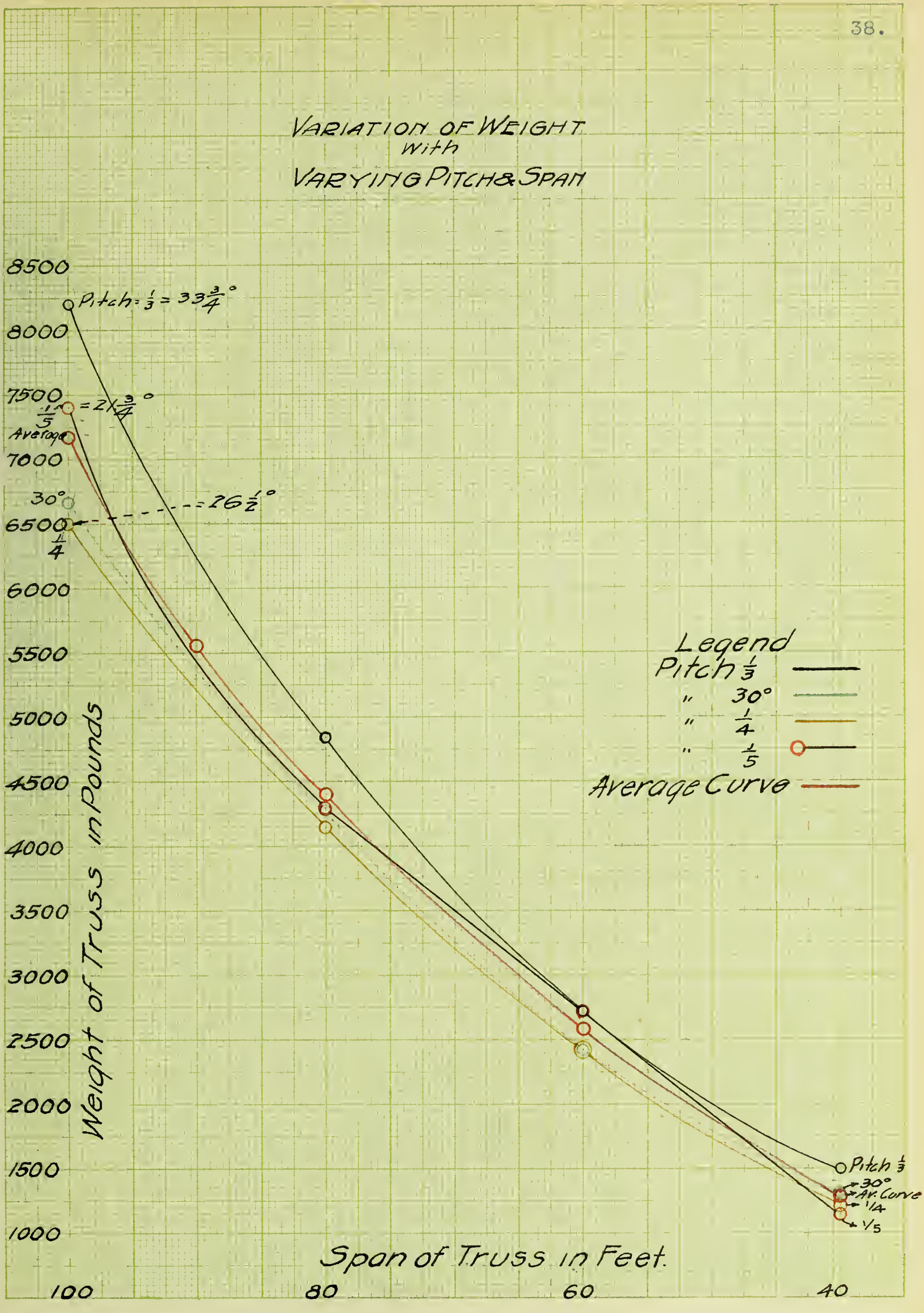
Economical Pitch
Span - 40'
C-C- 16'
1450
1400
1350
1300
1250
1200
1150
Weight of Truss in Pounds.
Probable Error
50 lbs.
Note: in the following curves, the red show Ilg & Wagner's results, except as noted.
22°
23°
24°
25°
26°
27°
28°
29°
30°
31°
32°
33°
34°
1/5
1/4
1/3
Pitch of Truss

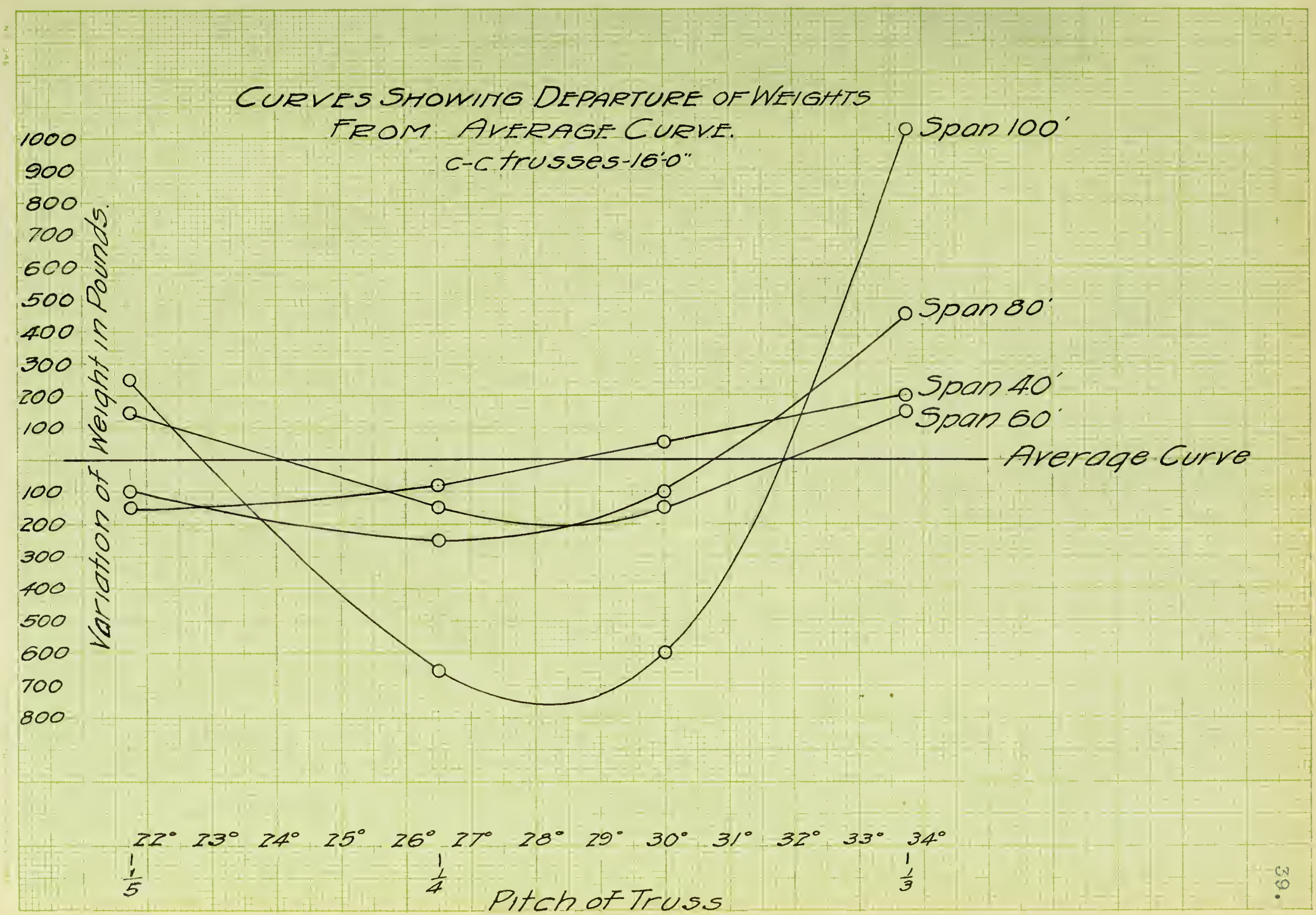

ECONOMICAL PITCH
SPAN 60'
c-c-16
2750
2700
2650
2600
2550
2500
2450
Weight of Truss in Pounds
50 lbs.
22°
23°
24°
25°
26°
27°
28°
29°
30°
31°
32°
33°
34°
1/5
1/4
1/3
Pitch of Truss



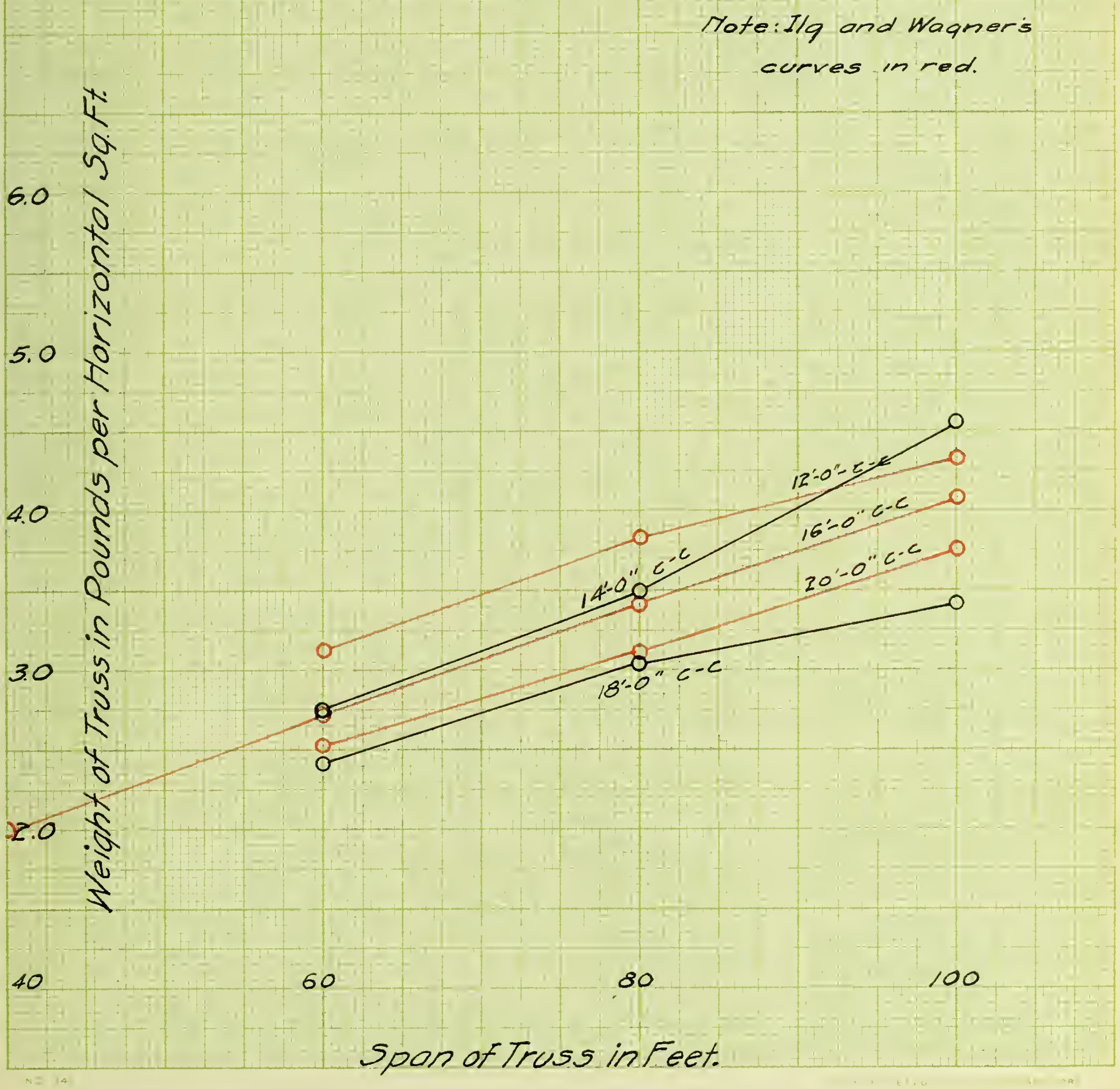
ECONOMICAL PITCH
SPAN 80'
C-C-16'
4800
4700
4600
4500
4400
4300
4200
4100
4000
Weight of Truss in Pounds
100 lbs.
22° 23° 24° 25° 26° 27° 28° 29° 30° 31° 32° 33° 34°
1/5
1/4
1/3
Pitch of Truss

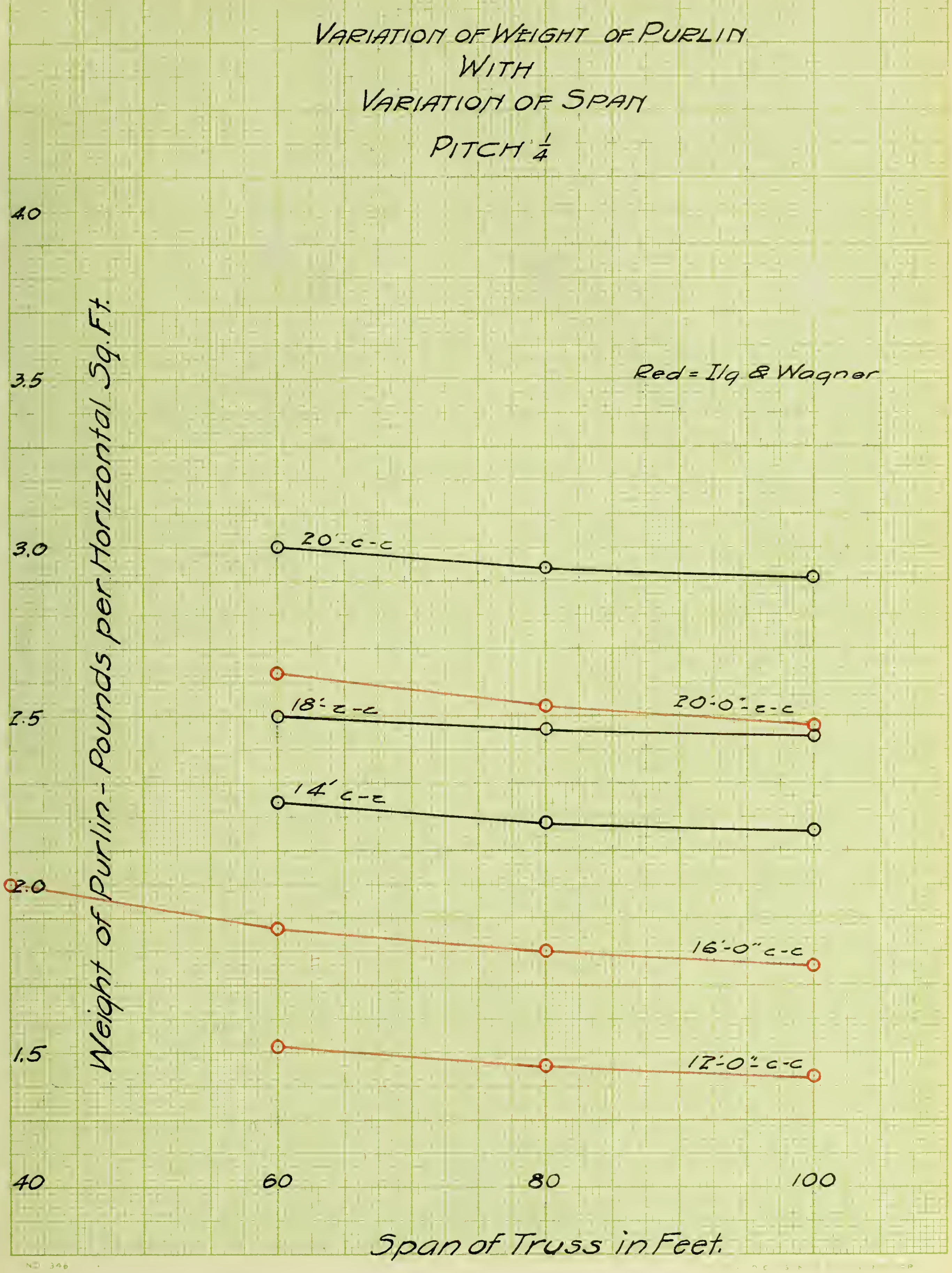

Economical Pitch
Span 100'
C-C-16'
8100
8000
7900
7800
7700
7600
7500
7400
7300
7200
7100
7000
6900
6800
6700
6600
6500
6400
100 lbs.
Weight of Truss in Pounds
22°
23°
24°
25°
26°
27°
28°
29°
30°
31°
32°
33°
34°
1/5
1/4
1/3
Pitch of Truss


VARIATION OF WEIGHT
with
VARYING PITCH & SPAN
Pitch = 1/3 = 33 3/4°
1/5 = 21 3/4°
Average
30°
26 1/2°
1/4
Legend
Pitch 1/3
" 30°
" 1/4
" 1/5
Average Curve
Weight of Truss in Pounds
8500
8000
7500
7000
6500
6000
5500
5000
4500
4000
3500
3000
2500
2000
1500
1000
Span of Truss in Feet.
100
80
60
40
Pitch 1/3
30°
Av. Curve
1/4
1/5

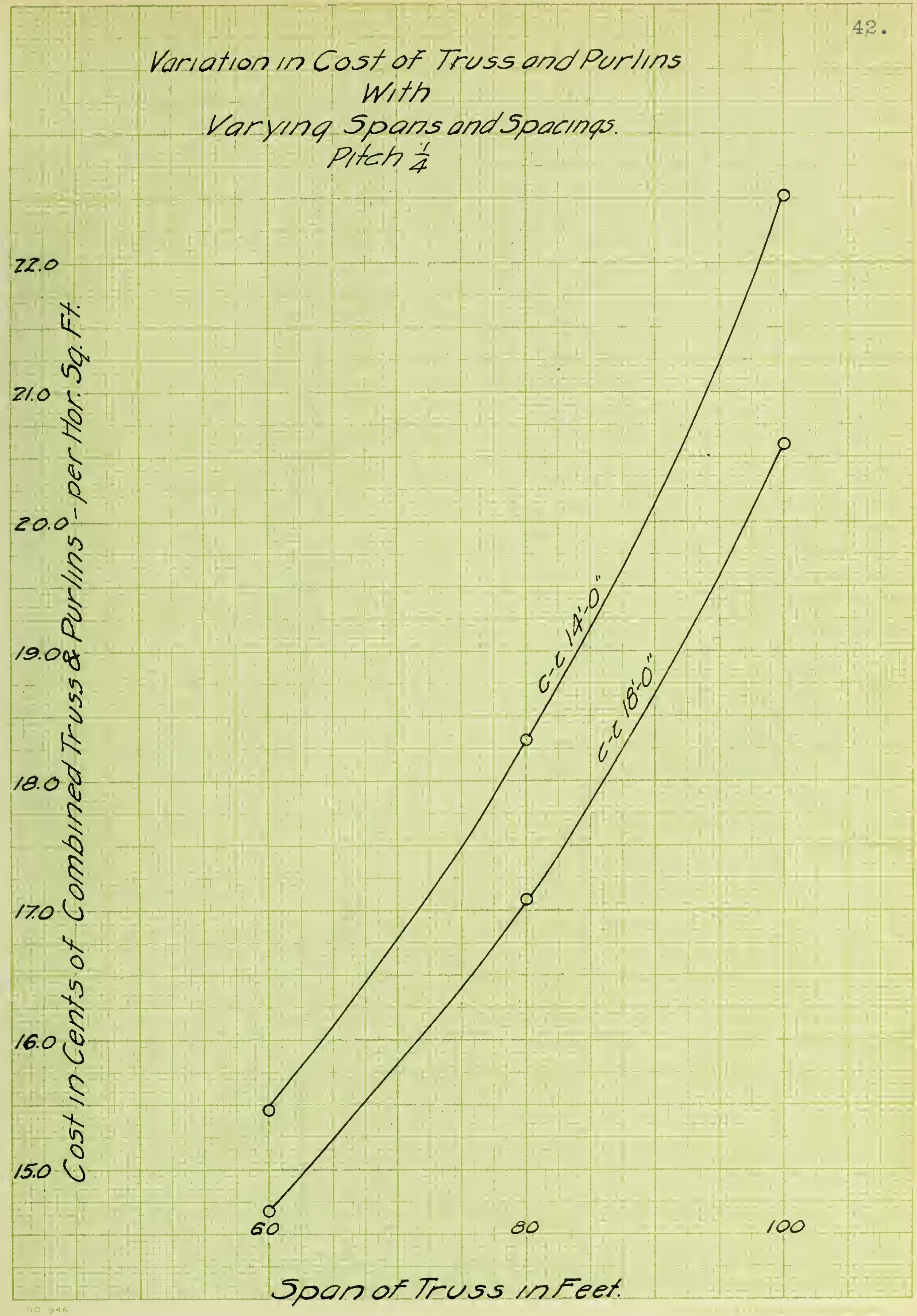

CURVES SHOWING DEPARTURE OF WEIGHTS
FROM AVERAGE CURVE.
c-c trusses-16'0"
Span 100'
Span 80'
Span 40'
Span 60'
Average Curve
Variation of Weight in Pounds.
1000
900
800
700
600
500
400
300
200
100
100
200
300
400
500
600
700
800
22° 23° 24° 25° 26° 27° 28° 29° 30° 31° 32° 33° 34°
1/5
1/4
1/3
Pitch of Truss

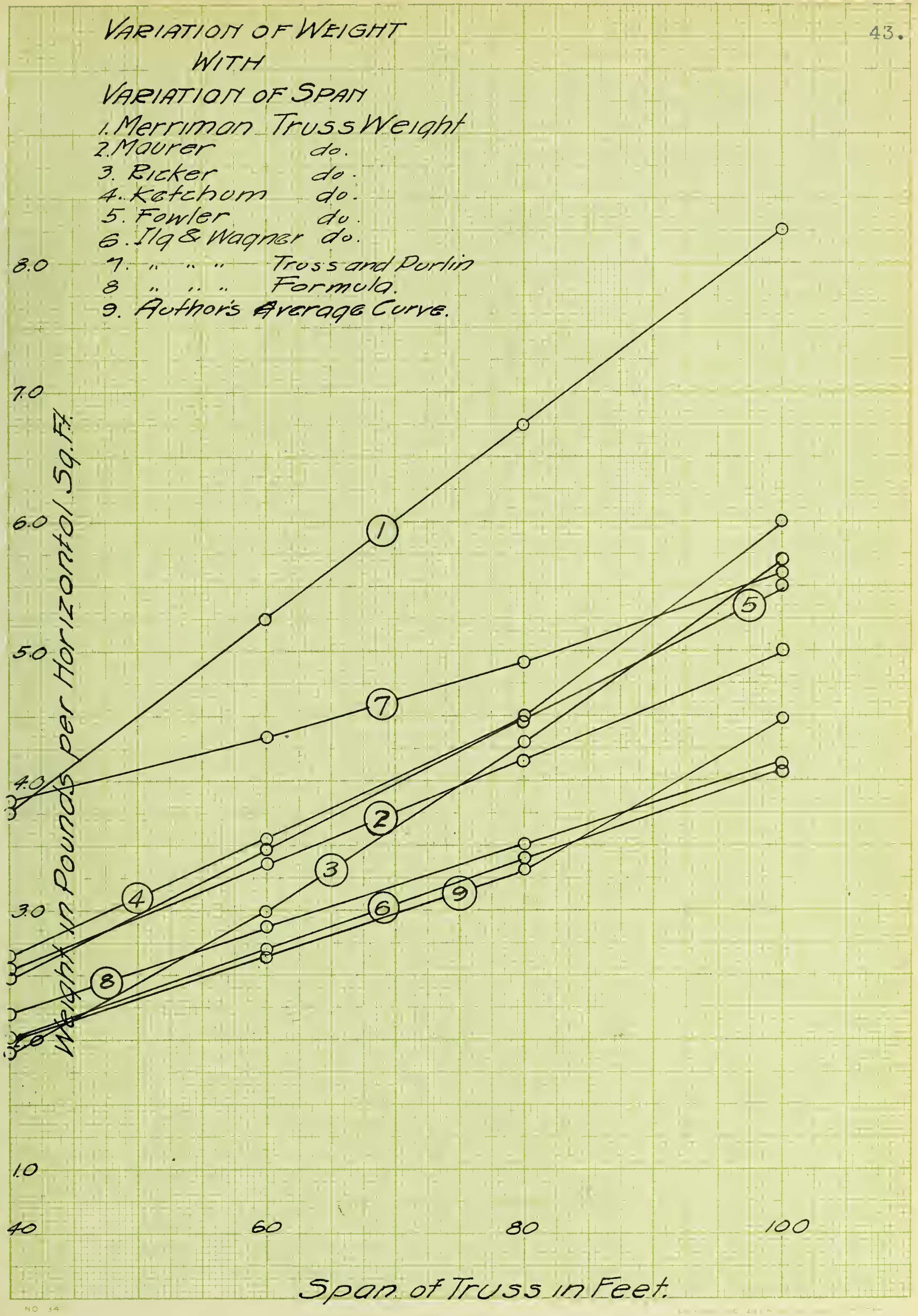
VARIATION OF WEIGHT OF TRUSS
WITH
VARIATION OF SPAN.
PITCH- $\frac{1}{4}$





VARIATION OF WEIGHT OF PURLIN
WITH
VARIATION OF SPAN
PITCH 1/4
Red = Ilg & Wagner
Weight of Purlin - Pounds per Horizontal Sq. Ft.
4.0
3.5
3.0
2.5
2.0
1.5
20'-c-c
18'-c-c
14' c-c
20'-0"-c-c
16'-0" c-c
12'-0" c-c
40
60
80
100
Span of Truss in Feet.


Variation in Cost of Truss and Purlins
With
Varying Spans and Spacings.
Pitch 1/4
Cost in Cents of Combined Truss & Purlins - per Hor. Sq. Ft.
22.0
21.0
20.0
19.0
18.0
17.0
16.0
15.0
c-c 14'-0"
c-c 18'-0"
60
80
100
Span of Truss in Feet.



Variation of Weight
With
Variation of Span
1. Merriman Truss Weight
2. Maurer do.
3. Ricker do.
4. Ketchum do.
5. Fowler do.
6. Ilg & Wagner do.
7. " " " Truss and Purlin
8 " " " Formula.
9. Author's Average Curve.
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
Weight in Pounds per Horizontal Sq. Ft.
40
60
80
100
Span of Truss in Feet.
1
2
3
4
5
6
7
8
9

Summary of Results.

Table XI.

Span- in Feet.	40	40	40	40	40	40	40	60	60	60	60	60	60	80	80	80	80	80	80	100	100	100	100	100	100	100	100
Pitch in Degrees	23	24	25-30	26	27	28	32	24	27	28	29	31	32	24	27	28	29	31	32	24	27	28	28-30	29	29-30	31	32-30
Merriman $w=\frac{3}{4}(1+\frac{l}{10})$	←			3.75			→	←		5.25			→	←		6.75			→	←			8.25				→
Maurer $w=1+\frac{l}{25}$	←			2.60			→	←		3.39			→	←		4.20			→	←			5.00				→
Ricker $w=l(\frac{240+l}{6000})$	←			1.87			→	←		3.00			→	←		4.26			→	←			5.68				→
Ketchum $w=0.89(1+\frac{l}{\sqrt{5a}})$	←			2.67			→	←		3.56			→	←		4.45			→	←			5.34				→
Fowler $w=0.05\times l+.05$	←			2.50			→	←		3.50			→	←		4.50			→	←			5.50				→
Ilg & Wagner $w=(1+\frac{l}{8\sqrt{a}})$	←			2.20			→	←		2.87			→	←		3.50			→	←			4.12				→
Author's Design	1.80	1.82	1.85	1.89	1.92	1.95	2.19	2.70	2.53	2.54	2.52	2.56	2.60	3.26	3.27	3.26	3.31	3.47	3.45	4.17	4.23	4.01	4.06	4.10	4.17	4.27	4.70
Aver. Curve, $w=-\frac{68}{l}+4.8-0.0471l+0.000505l^2$	←			2.04			→	←		2.67			→	←		3.42			→	←			4.47				→

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